HIGHWAY RESEARCH REPORT

CALIFORNIA PAVEMENT FAULTING STUDY

INTERIM REPORT

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STATE OF CALIFORNIA BUSINESS AND TRANSPORTATION AGENCY DEPARTMENT OF PUBLIC WORKS DIVISION OF HIGHWAYS	MATERIALS AND RESEARCH DEPARTMENT RESEARCH REPORT NO. M & R 635167-1

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STATE OF CALIFORNIA—TRANSPORTATION AGENCY RONALD REAGAN, Governor

DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS
MATERIALS AND RESEARCH DEPARTMENT
5900 FOLSOM BLVD., SACRAMENTO 95819

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Research Project No. 635167
Federal No. HPR D-3-32

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State Highway Engineer

Dear Sir:

Submitted for your consideration is a report on

CALIFORNIA PAVEMENT FAULTING STUDY

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Very truly yours,

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BFN:fp

Reference: Spellman, D. L., Stoker, J. R., and Neal, B. F., "California Pavement Faulting Study" State of California, Transportation Agency, Department of Public Works, Division of Highways, Materials and Research Department, Research Report 635167-1, January, 1970.

Abstract: Details of an investigation of faulted concrete pavements constructed on cement treated bases are presented. Various tests and measurements were made on the pavements and construction materials including strains, deflections, load transfer effectiveness across the undoweled joints, joint openings and movements due to temperature changes, slab curl, compressive strength, and petrographic and chemical tests. At some sites, movement of water at the slab base interface was determined by the use of colored tracer sands placed under the slabs several weeks prior to slab removal or coring.

Portions of pavement slabs were removed at fourteen sites in three different geographical regions -- Valley, Coastal, and Mountain. The sections, removed from the outer edge of pavement, were three feet wide and extended approximately three feet on either side of the joint. Ten of the joints thus opened were faulted in amounts varying from 0.10-inch to 0.30-inch. At each faulted joint, a buildup of granular material was found under the approach slab, and in some instances, under the leave slab as well. The buildup differential was approximately equal to the amount of faulting measured. There was no evident settlement or faulting of the cement treated base course. The major source of the buildup material was definitely identified as being material eroded from the cement treated base in one instance, and the untreated aggregate base from the shoulder in another instance.

It is concluded that the faulted condition of the joints was created by a buildup of material under the paving slabs at the joints. The buildup was caused by violent water action on available loose or erodable materials which were present beneath or adjacent to the slabs.

Abstract: (Continued)

To prevent or reduce faulting, more erosion resistant materials are needed at the surface of the base course and in the shoulders. In addition, consideration should be given to preventing the intrusion of water, or providing positive drainage.

Key Words: Cement treated bases, compressive strength, concrete, curling, faulting, instrumentation, pavement, pavement joints, strain gages, water action.

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The opinions, findings, and conclusions expressed in this report are those of the authors and are not necessarily those of the Bureau of Public Roads.

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CALIFORNIA PAVEMENT FAULTING STUDY

INTRODUCTION

The term "faulting", as used in this report, refers to the vertical displacement of concrete paving slabs at joints. As faulting progresses, the increased step-off at the joint affects the riding quality and other conditions of the pavement. When faults reach 0.12 to 0.15-inch, they become noticeable to motorists, and at about 0.20-inch, many drivers have been observed to stop and check their tires because of the "thump" they feel at each joint.

In the 1940's, Hveem(1) conducted an investigation into the causes of faulting and pumping of joints in California's PCC pavements. He concluded that "the basic cause and origin of all joint troubles, including mud pumping and faulting, is the volume change of concrete arising from variations in moisture and temperature." The volume change results in curling or warping of the slabs allowing alteration of the base support under passing wheel loads, especially when free water is present. His recommendation was that if slab movement cannot be prevented, the base material must be treated so as to "withstand the beating."

Since 1946 bases under California pavements have been treated, mostly with cement (CTB), but some with bituminous material (BTB). As recently as 1960, it was considered that the faulting problem had been reduced to a tolerable level. More recently joint spacing was randomized (13, 19, 18, 12 feet) to improve pavement riding quality, and joints were skewed (2 feet in 12, counter clockwise) to reduce the shock of sudden load transfer from one slab to another, thus preventing excessive stress on the interlocked faces of joints. In addition to these changes, the CTB was widened to extend one foot beyond the pavement edge. This change was made in part, to eliminate or reduce the possibility of edge pumping.

However, it became increasingly evident during the next few years that while the faulting problem was minimized, it was not completely solved by the above changes. A few pavements which had been performing well for a period of time, eventually developed considerable faulting. In addition, a few projects were found to start faulting in less than five years. During a general discussion with representatives of the Portland Cement Association in early 1967, the problem of early faulting was set forth. Since the problem was of national interest, it was proposed that the Association join with the State in an investigation, and that a research work plan be submitted to the Bureau of Public Roads for their approval and financing. This was done and with Bureau approval the investigation started in the spring of 1968. The Portland Cement Association generously offered their services at no cost to the State or Federal government, and their participation was welcomed.

FIELD INVESTIGATION - 1968

In cooperation with Portland Cement Association representatives, a work procedure was agreed upon which, it was felt, would best utilize the research capabilities available. The PCA would furnish instrumentation, equipment and the necessary manpower to obtain deflections, strains, joint openings, joint movements, slab curl, plate bearing tests, and share in the project engineering. The State would provide the equipment and manpower to remove portions of slabs, obtain samples of the roadway materials, do field and laboratory testing, provide field documentation, project engineering, and any other work found necessary during the investigation.

The pavements selected for investigation were as follows:

1. A portion of I-80 freeway through Sacramento not yet opened to traffic. Age at time of test was three months. This site was selected to familiarize the investigators with the testing procedure and to determine the necessary equipment and manpower needed to properly conduct the investigation. It would also provide strain and deflection information on a pavement not yet subjected to traffic.

2. A 6-lane freeway on Interstate 80 west of Sacramento near Vacaville. Although this highway had been open to traffic for only four years, there was some faulting up to 0.25-inch. Three slab opening locations were selected:

   A. The outside, or No. 3 lane at a joint faulted about 0.12-inch.

   B. The same joint as A, although unfaulted, on the inside (median) or No. 1 lane.

   C. Another joint in the No. 3 lane about 500 feet from A and B, which was faulted about 0.18-inch. The purpose of this opening was to provide replication of Site A.

3. A 9-year old four-lane freeway in the mountain region of I-80 near Truckee. This location was selected to compare deflection and strains at right angle and skewed joints, both of which were constructed on one project. Although some cores were to be taken, no slab openings were planned at this site.

4. A four-lane freeway on U. S. 99, south of Sacramento near Manteca. This pavement was 13 years old and was seriously faulted. One-way truck traffic was nearly 2000 vehicles per day. The CTB on this project was constructed to the same width as the pavement.

5. A four-lane highway on U. S. 99, south of Sacramento near Turlock. This pavement, though in use for 17 years and carrying approximately 2000 trucks per day, was not seriously faulted. The CTB on this project was also constructed to the same width as the pavement.

The last two locations were selected in the hope that they might reveal differences responsible for extremely variable performance where environment, construction materials, design, and traffic were very similar.

TEST EQUIPMENT AND INSTRUMENTATION

A semi-trailer unit was furnished by PCA and used to apply both static and moving wheel loads at the first three test locations. In addition, it provided the static load reaction necessary to conduct plate bearing tests.

Benkelman beams were used to measure deflections at and across joints under static load. Moving load deflections were measured by resistance bridge deflectometers referenced to cased rods driven into the subgrade. Load strains were measured by SR-4, Type A-9 gages cemented to the concrete surface and recorded electronically.

Movements of slab corners to temperature and moisture changes (curl) were measured by slip-pin deflectometers. Temperatures of top and bottom concrete surfaces as well as shaded and unshaded air were sensed by liquid filled tubes and logged on an automatic recorder. Initial joint openings were measured with an optical comparator and changes in openings were determined with a vernier caliper across brass reference plugs placed in the pavement surface at each side of the joint.

Due to other commitments, the PCA testing crew was unable to remain in California for the entire field investigation. Therefore, Benkelman beam tests at Sites 4 and 5 were made with the State's equipment. The truck had a shorter wheelbase than the PCA unit, but was loaded to the same 18,000-pound, single axle weight.

PROCEDURE

The normal procedure at locations where slabs were removed was to select test joints and install instrumentation on the first day, make tests and measurements on the second, then remove a portion of the pavement, complete testing and sampling, and repair pavement on the third day.

At each test site of the first three locations, three joints were selected for instrumentation and testing. These joints represented comparatively high, medium or low load transfer effectiveness based on data obtained from at least ten Benkelman beam deflection tests at successive joints. Load transfer effectiveness was computed using a formula developed by Teller and Sutherland(2).

$$E(\%) = \frac{2\,d_j'}{d_j + d_j'} \times 100$$

in which d_j' is the deflection of the unloaded slab, and d_j is the deflection of the loaded slab. If the deflections of the loaded and unloaded slabs are equal, joint effectiveness would be 100%. If only the loaded slab deflected, it would indicate no load transfer and the effectiveness would be zero.

After the joints were selected, the shoulder material adjacent to the joints was excavated to allow for installation of the deflectometer. Excavation was to neat lines to provide solid support for cover plates needed to protect the traveling public during the test period.

Measurements of temperature, slab curl, joint openings, strains, and deflections proceeded throughout the day according to a preselected schedule. In addition, elevations were determined along the pavement edge of two slabs to see if curl could be detected with temperature change. Rod readings were made to the nearest 0.01-inch with a self-adjusting level. Points were painted on the pavement for repeat measurements.

Portions of the concrete pavements were removed at locations 1, 2, 4, and 5. The size of the portions removed was three feet by six feet (approximately three feet on each side of the joint). Parallel saw cuts were made around the section to be removed to provide neat lines for repair and to facilitate final cutting with pneumatic pavement breakers. Joints were sealed with grease to prevent the entry of water during the sawing operations. Before final breaking, a lifting device was bolted to the slab into two drilled holes. The cement treated base was also cut through so that if sufficient bond existed, it would be lifted with the concrete. (However, this did not occur.) When the slab was isolated, it was carefully lifted out and set aside. A hydraulically operated front-end loader was found to be entirely satisfactory for this purpose.

After slab removal, the condition of the base surface was noted and documented, and samples of any loose material obtained. At Locations 1 and 2, the entire CTB section was removed and plate bearing tests made on the subbase at each side of the joint. Samples of the shoulders, base, subbase, and underlying materials were obtained before the opening was backfilled. At least three 5-inch cores were taken in adjacent slabs for determining relative values of compressive strength and modulus of elasticity.

TEST RESULTS

Most of the physical test data, which were only indirectly applicable to the conclusions, are shown in the Appendix starting on page 42.

OBSERVATIONS

Note: Throughout this report, reference is made to "approach" and "leave" slabs. These terms refer to the slabs on either side of transverse contraction joints. If traffic is considered as moving from left to right, the "approach" slab is on the left and the "leave" slab is on the right.

Location 1 - Sacramento

Since this was a new pavement, no significant findings were expected, nor were any found. However, it did serve the valuable purpose of familiarizing field personnel with the techniques of testing and slab removal before moving to locations under traffic. It was noted that there was no evident bond between the CTB and PCC (see Figure 1) and no cracks were found in the CTB. This site may be re-examined in the future, in which case, conditions before and after traffic can be compared.

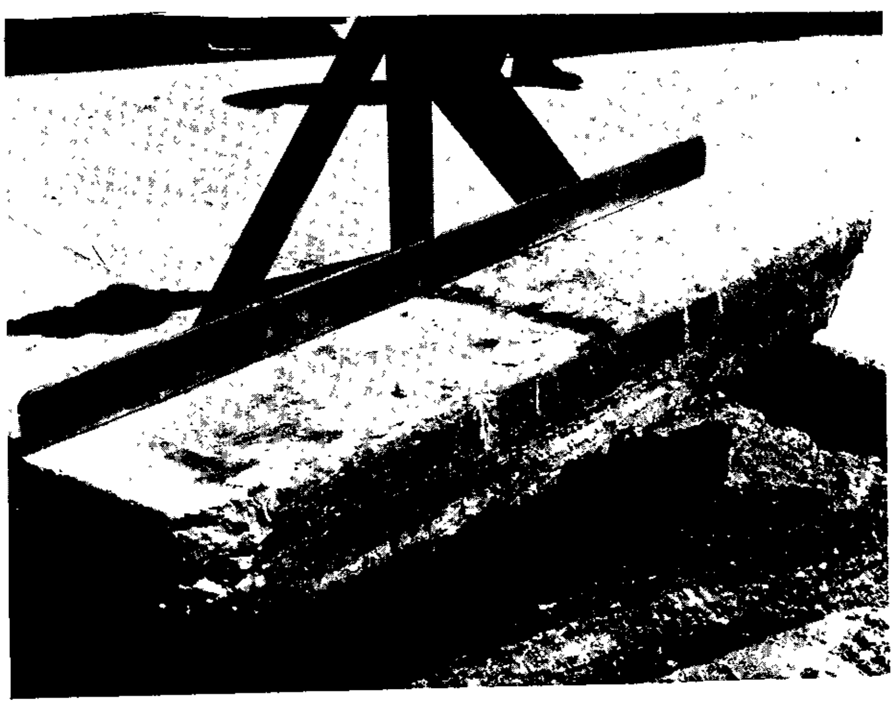

Figure 1

Location 2A - Vacaville, I-80

The outside, or No. 3 lane, was constructed with 0.75-ft. PCC over 0.33-ft. CTB, and carries the major portion of the 2000+ average daily truck traffic (one way). Although only open to traffic for four years, a few of the joints in this lane had faults of 0.20-in. or more. In general, faulting ranged from 0.05 to 0.15-in., and the joint selected for opening was faulted 0.12-in. (measured about one foot from the edge of the pavement). After opening the shoulders and installing gages, the site was left for a weekend, during which a moderate rain fell. On returning to the site on Monday, there was a fairly large quantity of granular material in the shoulder opening which apparently had been ejected from under the slab. This can be seen as the finer grained material in Figure 2.

Figure 2

Slab removal was accomplished without difficulty and with no apparent disturbance of the CTB. Some water from the sawing operation was found under the slab indicating a partial ineffectiveness of the grease in sealing the joint at this site. A small ridge of concrete and sandy particles was present directly under the joints. (A similar ridge was found at every joint where slab portions were removed, and may be due to compression failure when slabs are in an upward curled condition.) Figure 3 is an overall view of the site. A buildup of granular material can be seen in the foreground (which is the approach side of the joint), the ridge of concrete particles in the center, and the relatively clean but eroded CTB surface on the leave side of the joint. The layer of loose material covered about five square feet. The thickest layer was near the joint and tapered out within two or three feet. Most of the layer was composed of a brown silty material, but there were some coarser particles. It appeared that segregation had taken place as a result of water action with coarser particles being deposited near the joint and finer particles farther back. The coarest particles were along the outer edge and can be more readily seen in the close-up view, Figure 4. A tight crack was found in the CTB, but was not directly under the PCC joint. There was no faulting of the CTB at the crack.

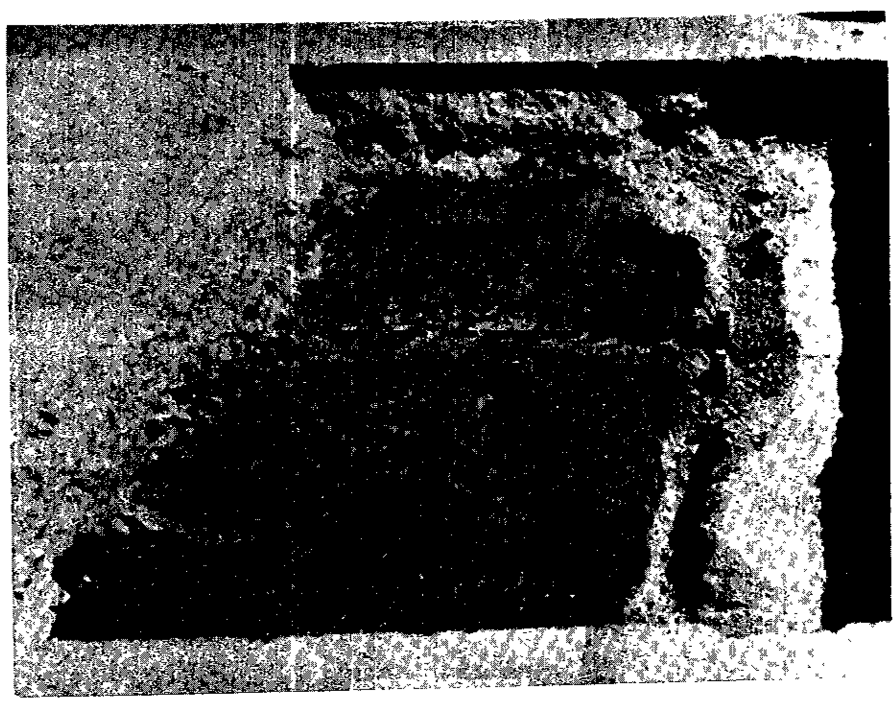

Figure 3

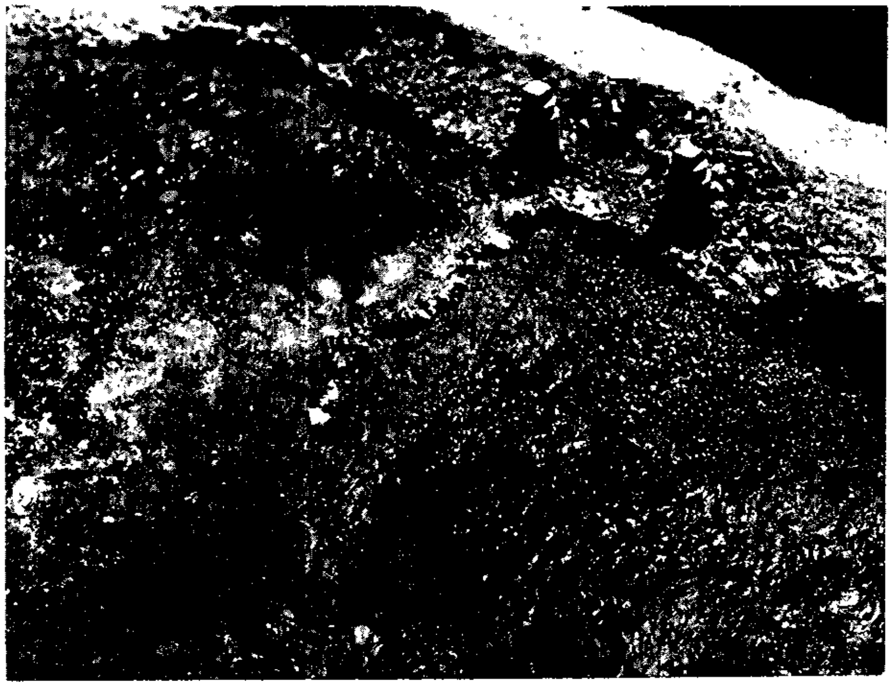

Figure 4

Location 2B - Vacaville

This site was on the inside, or No. 1 lane, and at the same joint as 2A. Pavement thickness in this lane is 0.67-ft. The purpose of the opening was to compare the unfaulted portion of the joint with the faulted. Figure 5 shows the condition of the CTB under the slab. A ridge of concrete and sandy particles was found under the joint, but there was no other sign of any buildup or movement of material. A tight crack was found in the CTB near the joint in this lane also.

Location 2C - Vacaville

This site was in the outer lane about 500 feet from 2A. The joint selected for opening was faulted 0.18-in. The buildup of material under the approach slab was very similar to that found at 2A, although a lightly greater amount. As seen in Figure 6, the coarser sized particles are a little father from the edge than at 2A. No crack could be found in the CTB.

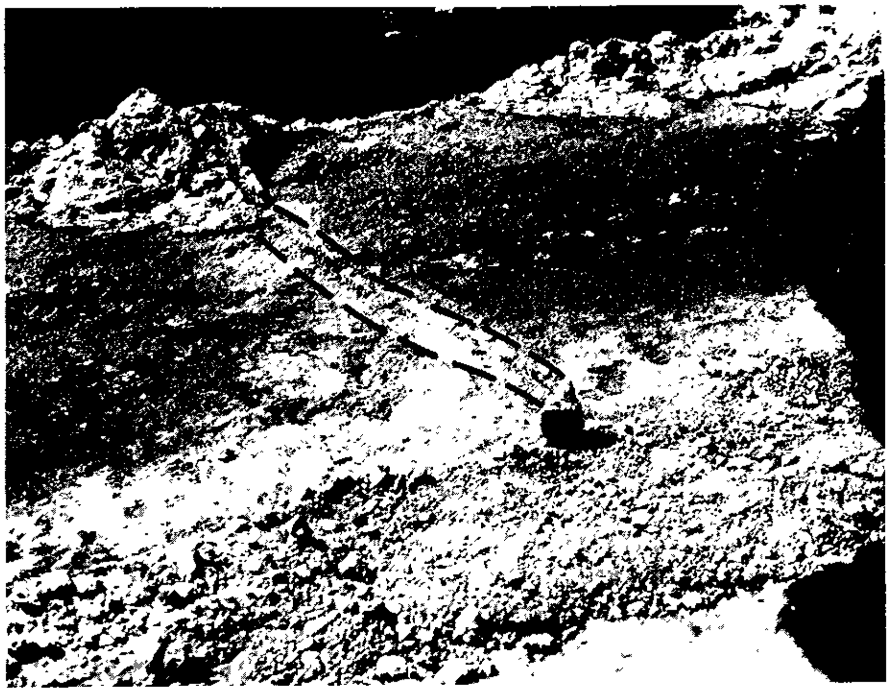

Figure 5

Figure 6

Location 3 - Truckee - I-80

There were no slabs removed at Truckee, but cores were taken to check bond between the PCC and CTB, and to determine the compressive strength of the concrete.

Location 4 - Manteca - U.S. 99

The pavement at Manteca was quite badly faulted, ranging from 0.10 to 0.30-in. with many joints faulted 0.20-in. The joint selected for opening was faulted 0.30-in. The buildup of granular material under the approach slab also measured about 0.30-in. Figure 7 shows this buildup on the right and the relatively clean but somewhat eroded CTB on the left. Small channels made by water are also clearly evident. The lighter areas are the coarser sand particles which were faster drying. Most of the buildup was brownish silt-like material. There was evidence that a crack once existed in the CTB, but the space had filled up completely with fine material. A 12-inch core was drilled through the crack and the core removed without breaking.

Figure 7

Location 5 - Turlock - U.S. 99

While a few of the joints at Turlock were faulted as much as 0.20-in., most were in the 0.10 to 0.15-in. range. The joint selected for opening was faulted about 0.13-in. The exposed surface after slab removal was very similar in appearance to that at Manteca, but the quantity of buildup was much less. The CTB also had a crack which had filled with fine material, but a core taken through the crack broke apart on removal. A small deposit of granular material was found under the crack in the CTB. The joints on this project were sealed during construction and the seal appears to have been reasonably maintained.

DISCUSSION

Although strains and deflection caused by heavily loaded trucks are of importance in concrete pavement design, the results obtained during this investigation did not show any definite correlation with faulting of joints. Deflections and strains were found to be greatly influenced by the variables of slab temperature and curl, speed of the moving load, and distance of the load from the pavement edge. Load transfer due to aggregate interlock also varied with changes in pavement temperature with some joints indicating up to 100% load transfer when the pavement was warm, but was reduced to 50% effectiveness, or less, when the pavement was cool. (See Appendix, Table 2.)

Unfortunately, the running of levels along the edge of pavement did not provide satisfactory information on slab curl due to temperature changes. Either the rod could not be replaced on exactly the same spot, or the readings were not made with sufficient accuracy.

A profilogram section is shown on page 66 of the Appendix to provide a graphical illustration of a faulted pavement. This profilogram was obtained at the Vacaville test location with California's 25-foot wheel base, truck-mounted profilograph. Longitudinal curl of the slab and joint faulting can be readily seen. It can also be seen that the magnitude of faults is much less at the inner wheel track than at the outer wheel track.

The compressive strengths and elastic modulii as determined from the concrete cores, were found to be more than adequate at all test sites. The plate bearing tests at Sacramento and Vacaville indicated sufficient bearing capacity of the subbase materials. There was no significant difference between tests made on each side of the joint. In addition to plate bearing tests, R-values* were determined from subbase samples taken at each test site. These tests also indicated adequate stability of the supporting material.

* R-value is a coefficient representing the shearing resistance to plastic deformation of a saturated soil at a given density, and is measured by the stabilometer.

The findings from the slab openings at Locations 2, 4, and 5, can be summarized as follows:

1. There was no measurable bond between the PCC and the CTB. All PCC slabs lifted clear of the CTB.

2. There was a buildup of material of granular nature under the approach side of a faulted joint. No such buildup occurred under the one unfaulted joint examined.

3. All of the faulted joints had a layer of brown colored silt-like material under the approach side of the joints. Since there were definite indications of asphalt in the material, it is believed to be the source of the brown color. (Asphalt, usually a medium curing product, is used as a membrane cure for the CTB.)

4. The faulted joints had a layer of coarser grained material on top of the silty material, but grading differed somewhat at the various locations; i.e.,

   a. Vacaville sites - well graded, 3/8-inch to No. 100
   b. Manteca site - mostly No. 30 to No. 100
   c. Turlock site - similar to Manteca site, but not enough material to obtain a representative sample.

   (There was no coarse material of any consequence used in the construction of the base, subbase, or shoulders at the Manteca and Turlock sites.)

5. There were indications of water movement between the PCC and CTB as evidenced by (a) visible water flow marks or channels on the leave side of the joint and in the deposited material under the approach side, and (b) visible sorting of the sand under the approach side.

6. There was evidence of slight spalling along the bottom edges of the concrete slabs at both sides of the joints.

7. There was a brown stain on the lower portions of the concrete interfaces of the joints at all locations. The discoloration appeared to stop at the bottom of the saw cut or, in the case at Turlock, the bottom of the joint sealant. Evidently some material is being pumped up into the joint.

8. The CTB was not always visibly cracked under the pavement joint. Where cracks had occurred, they were tight with no evidence of faulting.

9. The surface of the subbase appeared in all cases to be undisturbed although at Turlock, a small deposit of granular material was found directly under the crack in the CTB.

10. At Vacaville Sites A and C, there was evidence that some of the aggregate base shoulder material was being lost. Voids were found in the aggregate base at the edge of the slab.

    Also observed were depressions of the top surface of the asphalt concrete shoulder material adjacent to the concrete at the two faulted joints. Similar depressions can be observed at many other joints on this and other projects.

Not all of the desired answers were found during the investigation. The source, or sources of the granular buildup under the slab could not be determined. Five possible sources (or combinations thereof) were considered:

1. Aggregate base in the shoulder
2. Top surface of the cement treated base
3. Contamination between PCC and CTB left during construction
4. From outside, through the joints
5. From abrasion and spalling in the PCC joint

Although at Vacaville there were indications of shoulder material loss, definite identification could not be made from the buildup samples since the aggregate source for both the shoulders and the CTB was the same. The same problem was encountered at Manteca and Turlock where the subbase, base, and shoulders, were constructed with fine grained sandy material from local borrow sites.

After thorough study of the results of work done in 1968, it was decided that further field work was necessary. A new work plan was formulated to continue field study in the spring of 1969 to provide answers to the following questions:

1. Were the joints which were opened for inspection typical of other faulted joints throughout the state?

2. What is the source or sources of the granular material found built up beneath the slabs?

3. How is the material transported?

4. Why does the buildup occur at some joints but not at others in the same vicinity?

FIELD INVESTIGATION - 1969

To at least partially answer the question about geographical influence on faulting, the 1969 phase included the coastal and mountain regions. Along the coast, three contiguous projects on U.S. 101 between San Luis Obispo and Santa Maria were selected. While all three projects had been open to traffic for about 13 years, two of the three were badly faulted and the third (when surveyed in the fall of 1968) had only minor faulting and was relatively smooth riding. Three locations were also selected for study on I-80, east of Sacramento, at the 5000-6000 foot elevation. Faulting on these 6 - 10 year old pavements had not yet reached the serious stage although a few joints were faulted to 0.20-inch. Transverse contraction joint variables at the six locations included sealed and unsealed, 15-foot spacing at right angles; 15-foot spacing with skew, random spacing with skew; and faulted and unfaulted joints. Those in the coastal region were unsealed, sawed at right angles, and spaced at 15 feet.

To trace the movement of material under the slabs, special sands were placed around the joints at sites preselected for opening. Three distinctive colors (pink, green, and purple) of aquarium sand were selected for this purpose. Approximately one-half of each color was crushed, then blended back to obtain a better graded product. These sands were then placed in about 0.5-lb. quantities through 2-inch core holes. At most locations, one color was placed in the shoulder at the bottom of the slab adjacent to the joint, and the other colors were placed through the slab 6 to 8 inches on either side of the joint and about 2 feet from the shoulder. For possible future study, more joints were "treated" than were proposed for opening during this investigation.

Tracer sands were placed during the week of March 24 and slabs were removed during the weeks of May 5 and 12. Rainfall in the areas during this period is shown on page 63 of the Appendix.

The procedure at each location was as follows:

1. Install gage plugs across at least three consecutive joints to measure movement due to temperature and moisture changes.

2. Measure joint opening during hottest part of day. (At Location 1, shoulder openings were made for this purpose, but found unsatisfactory due to dirt and other debris adhering to the joint interfaces. At other locations, satisfactory measurements were obtained through 10-inch core holes at the joints.)

3. Make elevation measurements along the edges of two consecutive slabs to determine curl. Nails were to be driven into the concrete as reference points to improve the repeatability of subsequent measurements.

4. Seal all joints and cracks with melted paraffin before making saw cuts.

5. Cut and remove slab.

6. Observe and document all details of the exposed surface.

7. Sample any buildup of material found on top of the CTB.

8. Obtain PCC and CTB cores, and sample subbase material.

OBSERVATIONS AND COMMENTS

Location 1
Pismo Beach - U.S. 101

In general, faulting at Pismo Beach ranged from 0.15 to 0.30-inch, and the joint selected for opening was faulted 0.22-inch. There were no unfaulted joints available for investigation and comparison. The opened site is shown in Figure 8. The paraffin proved to be completely effective in preventing the entry of water during sawing. Some of the wax even ran through the joint and spread out under the leave slab indicating the presence of a void. The green sand placed under the approach slab was entirely undisturbed and may be seen in the center of Figure 8 as a little mound. There was also no movement of the pink sand which had been placed in the shoulder. However, the purple sand which was placed under the leave slab was completely gone from the spot at which it had been placed. A small amount was found at the joint location, but most had been moved farther back and deposited under the approach slab in an area about one foot wide and two feet long. There were no indications of side or circular forces being exerted on the sand.

Figure 8

For the first time, buildup was found on both sides of the joint, although thicker on the approach side. Figure 9 shows a vertical cut through fairly loose material to a solid base. The original surface plane of the CTB could not be definitely established however, and some of the loose material may have been left during construction. The depth of the material on the left is about 3/4-inch and on the right, 1/2-inch, the differential being approximately equal to the measured fault. Although there was very little coarse material in the buildup, the same segregation or sorting of particles was found here as at the previous openings of faulted joints.

Figure 9

No crack was found in the CTB. Attempts to obtain cores failed due to weakness of the CTB. (Normally, cores can be obtained if the compressive strength is about 200 psi or greater.) Tests indicated the cement content of the CTB to be approximately 3%, which probably is not sufficient for the fine-grained base materials used.

Petrographic and chemical tests were also made on samples of the buildup and untreated shoulder materials. When water was added, almost a third of the buildup material from under the slab could be floated off as dark brown clay-sized particles. Found in the remainder were a few iron fragments and numerous brown sandy lumps which were calcareous. The iron fragments may possibly have come through the joint from the pavement surface. Most of the particles making up the sample were similar in composition to the shoulder and CTB materials, both of which were from the same source. Strong traces of asphalt were found which very likely accounted for the brown coloring previously referred to. From a calcium oxide determination, the cement content of the granular material was calculated to be about 1%. Based on this low cement content in comparison with those found in buildup at other sites, the shoulder material would appear to be the major source of the buildup at this site.

Location 2
Russell Turn - U.S. 101

This pavement was badly faulted and had numerous cracked slabs. The site selected for opening was typical of the project; the joint faulted 0.30-inch and the leave slab cracked 5-1/2 feet past the joint. In this case, concrete removal was extended longitudinally all the way to the crack. The additional length and weight did not create any problems of removal.

The tracer sands were placed somewhat differently at this location. Pink sand was placed in the shoulder, green sand directly through the joint, and a purple sand under the leave slab 5 feet from the joint and only 6 inches from the crack. Removal of the slab revealed that considerable activity had taken place since the sands were placed. Several particles of the pink sand from the shoulder were found as much as one foot in from the edge of the pavement under both the approach and leave slabs. The green sand placed at the joint was widely scattered in all directions. The purple sand had tended to migrate more toward the shoulder, but some had moved all the way back to the joint, a distance of about 5 feet. The directions of sand movement indicated a circular or swirling water action. The absence of water channels in the exposed surface (Figure 10) might also confirm this type of action. The water movement at this site may not be typical of movement which takes place under uncracked, full length slabs, however.

The layer of granular buildup was quite thick and again extended under both slabs. Petrographic tests on the samples identified the material as having approximately the same composition as the base and shoulders. Some iron fragments and glass beads were found and can reasonably be assumed to have entered through the joint. (Glass beads are used in traffic striping paint.) Tests indicated the cement content of the loose material to be between 5% and 7%, and that of the CTB as about 3%. There was sufficient strength of the CTB (220 psi) to obtain cores, however.

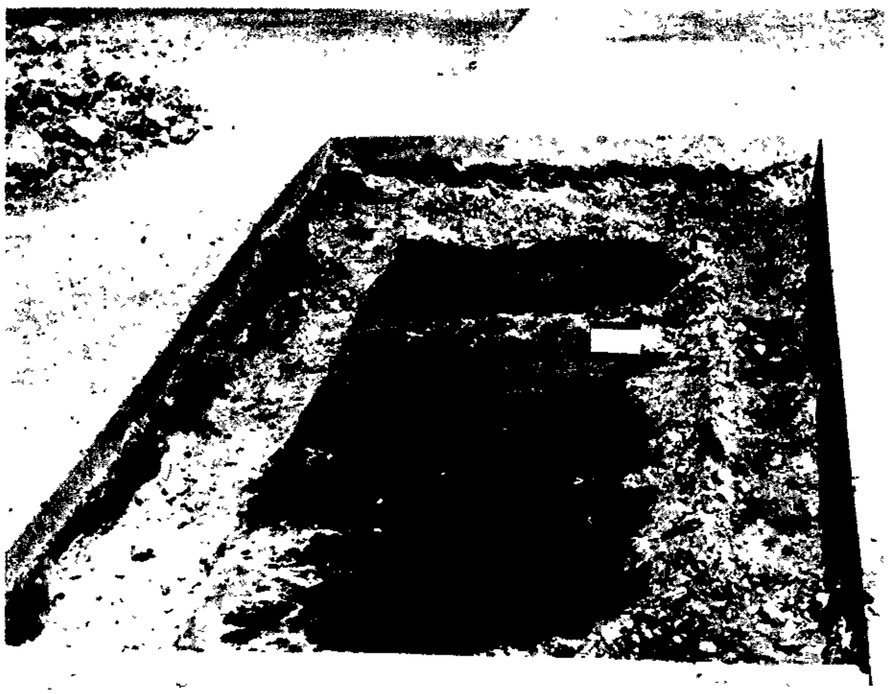

Figure 10

Location 3
Nipomo - U.S. 101

When originally selected in the fall of 1968, this project had only minor faulting ranging from 0 to 0.15-inch. By March 1969, however, faulting had increased significantly with many joints faulted 0.30-inch and none found with less than 0.10-inch. This change can probably be attributed to the heavier than normal rainfall that occurred during the winter, although only one other project was observed to have a similar increase in faulting during this short period.

It was planned that an unfaulted joint would be opened at this location as well as a faulted one. Since at the time of investigation there were no unfaulted joints, one with the least faulting available (0.10-inch) was chosen along with one faulted 0.30-inch. At the relatively unfaulted joint, only a few particles of the pink tracer sand placed in the shoulder were found under the slab. These were mostly under the leave side and within about 6 inches of the edge. The green sand placed under the approach slab had not moved at all. The purple sand placed under the leave slab was well scattered but had moved transversely more than longitudinally. None had crossed the joint to the approach side. A buildup of loose material was found on both sides of the joint and measured about 0.20-inch thick on the approach side and 0.10-inch on the leave side. As may be seen in Figure 11, there are no dominant longitudinal water channels. A tight crack was found in the CTB. The cement content of the buildup is not available, but that of the CTB was approximately 6%.

Figure 11

At the 0.30-inch faulted joint, the melted paraffin followed distinct channels under the leave slab (see Figure

12). The buildup under both slabs appeared to be quite thick and similar to that at Location 1. As was the case at Pismo Beach, however, the original surface plane of the CTB could not be definitely identified. Some of the pink sand placed in the shoulder had moved a few inches under the pavement, and again, mostly under the leave slab. The green sand from the approach slab was somewhat scattered with the greatest movement being toward the shoulder. The purple sand from the leave slab was almost entirely displaced to the approach portion. Some particles were found on the approach side as much as three feet from the joint. The position of these sands strongly suggests some circular motion of the water. There was also a tight, but unfaulted, crack in the CTB at this site.

Figure 12

The shoulder material at this location was a local native sand and was distinctively different from that used in the CTB. It had also been cement treated to improve its supporting value. With the help of these factors, the buildup samples were identified as being almost exclusively from the CTB under the slab. The color and composition of the CTB materials were different from those of any other materials used in the roadbed. The cement contents of the buildup, treated shoulder, and CTB were all about 4%.

Locations 4 through 6 are located in the mountainous area of Interstate 80. These locations are all subject to heavy snowfalls, subfreezing temperatures, the heavy use of deicing salts mixed with sand, and heavy tire chain traffic. The joints are sealed to exclude the ready intrusion of water and debris.

Location 4
Prosser

The joints at the site on I-80 were sealed, skewed, and spaced at 15-foot intervals. Faulting ranged from 0 to 0.15-inch. One unfaulted joint and one faulted 0.12-inch were selected for opening. Shoulders, as well as base, were cement treated.

At the faulted joint, no movement of the tracer sands had occurred during the seven week period after placement. (See Figure 13.) The buildup under the approach slab was

Figure 13

not extensive and was different in appearance and composition from that observed at all other sites previously examined. The material was very fine grained and tended to cling together to form lumps when dry. Tests indicated over 5% cement in the buildup, or about the same as found in the CTB. On each side of the joint, there were areas of weakness in the treated material of both the shoulders and the base. Some of the material was soft enough to be readily scraped away.

When the slab was lifted at the unfaulted joint, the CTB broke loose around the edges and remained bonded to the PCC (see Figure 14). Since the CTB at this opening had not been isolated by the pavement breaker, considerable bond strength had to exist. No measurement of the bond was obtained however. The asphalt curing membrane, where visible, was still in a plastic or tacky condition.

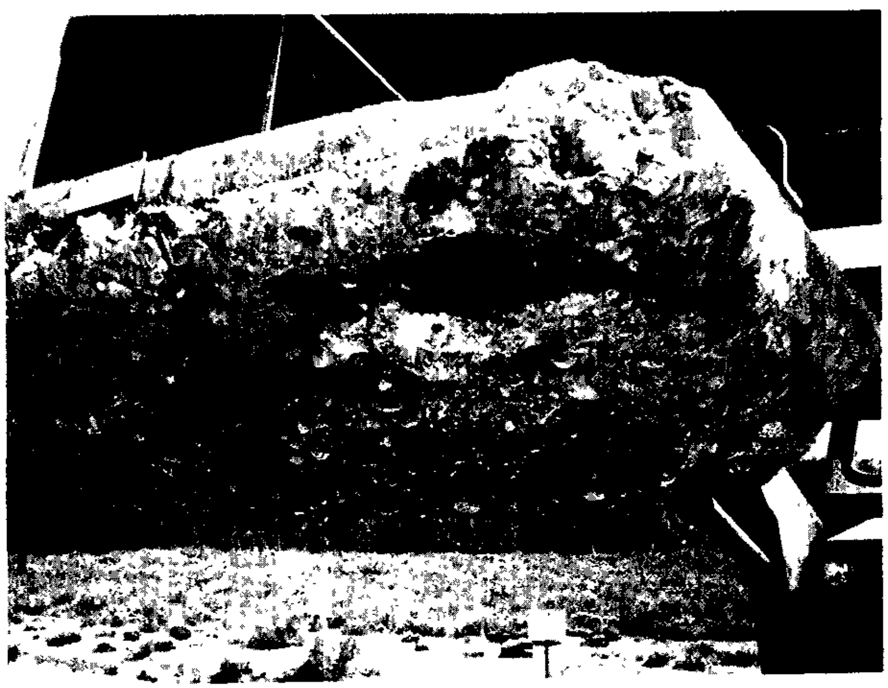

Figure 14

Location 5 - Yuba Gap

The joints on this project were sealed, skewed, and randomly spaced. The amount of faulting varied considerably ranging from 0 to 0.20-inch. At the site selected for investigation, four consecutive joints were alternately faulted and unfaulted. One unfaulted joint and one faulted 0.15-inch were opened.

At the unfaulted joint there was no apparent buildup (see Figure 15), and the tracer sands had not moved. In fact, most of the tracer sand adhered to the sides of the core hole and was removed with the slab. The CTB was so weak that cores could not be taken.

Figure 15

While there was some buildup at the faulted joint (Figure 16), the surface plane of the CTB was difficult to identify. As seen in Figure 17, the CTB was so cohesionless it could be removed with a shovel. No movement of the tracer sands had taken place.

Figure 16

Figure 17

Tests on the buildup material indicated a cement content of approximately 7%, while the CTB at both the faulted and unfaulted joints had only about 3 to 4% cement. Nothing was found at the site or in the project records which would explain why the CTB had no cohesion, although low strengths might be expected with the relatively low cement content. Where concrete cores were taken at adjacent slabs, the CTB condition varied from poor to good. The similarity of conditions at both joints makes it difficult to explain why one joint would fault and an adjacent one not fault. A possible explanation is that the unfaulted joints did not crack open and "work" as contraction joints during the first few years of pavement life. In other words, load transfer efficiency was higher at the unfaulted joints.

The buildup material could not be sufficiently identified to positively determine the source. The appearance and composition were similar to that at the Prosser location - fine grained particles tending to cling together when dry. Both the base and shoulders were cement treated although the cement content was less than that found in the buildup material.

Location 6 - Boca

This location was part of the same project as Location 4 - Prosser, although about two miles away. The pavement design was the same, but the site was selected because of the different terrain - a slight cut in a narrow canyon as opposed to a small fill in a large open valley. After exploratory cores at both faulted and unfaulted joints were taken, however, it was decided that removing slabs would not supply additional worthwhile information.

Several cores were drilled, both through the joints and at the middle of slabs. At all locations, the CTB was extremely weak. As seen in Figure 18, the crack interfaces of cores taken through the joints were quite heavily coated with a granular material. Although the joint interfaces at all locations had been stained, none of the others had such a heavy coating. The presence of cement in the coating was indicated by phenolphthalein tests, but calcium oxide content was not determined.

Location 7 - Santa Barbara

This location was not included as part of the finalized work plan, but was added after completion of the planned work because of special conditions known to exist. This PCC pavement was constructed in 1957 on a bituminous treated subgrade

Figure 18

and had untreated shoulders. Joints were sawed at right angles and at 15-foot spacing. During the winter of 1968-1969, the faulting on the project almost doubled in magnitude with numerous faults measuring 0.30-inch.

The investigation was very limited in scope with only three cores taken - two through approach slabs near faulted joints and one in the shoulder. It was found that about one inch of the BTB adhered to the concrete, then a separation occurred. At this separation, a "mud" lens was present, then the remainder of the BTB. This mud lens extended to the shoulder and was of approximately the same vertical dimension as the fault. The buildup material was identified as being of the same composition and appearance as the shoulder material.

FAULTING SURVEY

In order to determine the extent of faulting, a limited Statewide survey was made of rural PCC pavements constructed since 1960. The survey consisted of selecting two or three random locations on each project and measuring the faults at approximately ten consecutive joints. Many of the urban highways were driven and conditions observed, but measurements were not taken due to heavy traffic conditions.

Out of the total of 96 projects surveyed, 11 were less than two years old and were unfaulted. Of the remaining projects (3 to 8 years old), 12 had some faulting greater than 0.10-inch and four greater than 0.15-inch. A total of 43 of the 96 projects had some faulting greater than 0.05-inch.

Generally, only the outside or truck lane joints are faulted, with the greater magnitude at the outer edge and only a minor amount at the inner edge. There are a few exceptions, especially near urban areas where there are three or more lanes in each direction and a heavy concentration of truck traffic. It was also noted that faulting was usually less in urban areas. While the survey in these areas was very limited, it appears that faulting is also less where fast and complete drainage, such as curb and gutters, was provided. In urban areas there are more paved areas which also decrease the chance for water to get under the pavement.

There did not appear to be any reduction in faulting on the high side of the superelevation of curves to the left. This was studied rather closely since free water under the slab is considered a necessary element for faulting to occur, and it was thought that water might drain faster at these locations and be less available under the pavement on the high side.

Other factors studied briefly during the survey were upgrade versus downgrade faulting, geographical region influence, and the amount of truck traffic. The degree of faulting on upgrades appeared to be slightly greater than on downgrades, but the difference is not considered significant. Pavements in all areas of the State surveyed were found to be about equally subject to faulting. The number of trucks on almost all portland cement concrete highways is considered sufficiently large to cause faulting. One unfaulted pavement about

11 years old was found that carried a large amount of commuter traffic but virtually no trucks.

Very few joints can be found, even on the oldest pavements, which are faulted more than 0.30-inch. This is not necessarily because faulting is self-arresting, but rather because remedial action has been considered necessary. The remedy usually consists of grinding or overlaying, with the occasional use of mudjacking. On one project, it was noted that after severe faulting was eliminated by grinding, approximately the same degree of faulting reoccurred within two years. This is considered an extreme case, however, as a number of projects are still relatively smooth riding several years after grinding.

During reconstruction of an old freeway section in 1960, two eastbound lanes of the old highway became westbound lanes in the new. The original faulting (about 0.25-inch) in these lanes has now reversed and the joints are again faulted up to 0.20-inch in the direction of present traffic.

To check on the progression of faulting, test sections have been established throughout the State. Both faulted and unfaulted pavements have been selected and measurements made at 25 consecutive joints at random locations. Repeat measurements will be made at approximately 3-month intervals to determine changes in faulting. Since practically all of California's rainfall occurs between September and May, it is expected that the largest increases will be during this period.

SUMMARY

Early faulting of PCC pavement joints is considered to be a significant problem in California. On a few projects less than five years old, faulting has developed sufficiently to affect riding quality of the pavement. Almost half of the projects constructed during the past nine years have started faulting and there is no reason to expect this trend will change unless corrective design and construction methods are developed and implemented. Corrective maintenance action, whether by grinding, mudjacking, or overlaying, is expensive and not necessarily a permanent solution. The necessity of performing such maintenance should be eliminated if at all possible.

During the investigation reported here, a total of 14 openings were made at pavement joints in three different geographical regions -- valley, coastal, and mountain. Ten of the joints were faulted in amounts varying from 0.10-inch to 0.30-inch. At each site, a section of concrete three feet wide and approximately three feet on either side of the joint was removed. At each faulted joint, a buildup of granular material was found under the approach slab and in some instances under the leave slab as well, though to a lesser degree. The buildup differential was approximately equal to the amount of the fault. There was no evident settlement or faulting of the cement treated base.

At several locations, tracer sands were placed under slabs near the joints and under the shoulder pavement prior to opening. Upon exposure, definite evidence of strong water action was found under the leave slab but less action under the approach slab. Small channels caused by rapid water movement were evident under the leave slab at some sites. At one location there were indications that the water may have moved in a circular path, or at least involved strong movement in a direction transverse to the centerline of the pavement, in addition to longitudinal movement.

Various tests were made on the pavements and construction materials at the sites being investigated. These included strains, deflections, load transfer effectiveness across joints, joint openings and movements due to temperature changes, slab curl, compressive strength, petrographic and chemical tests. Many of the test results have not been of particular value in determining the cause of faulting. By petrographic and chemical tests, however, the source of the buildup at two sites was identified -- at Nipomo as the cement treated base, and at Santa Barbara as the shoulder material. The shoulder is also suspected of being the source of the buildup at Pismo Beach. Unfortunately, on most of the projects investigated, the construction materials used in the base and in the shoulders were from the same source or were of similar appearance and composition, so that the source of the buildup could not easily be identified. The relatively high cement content (based on calcium oxide determinations) of the buildup material at most sites strongly suggests that the CTB is a major contributing source. A small amount of the material also comes through the joint from the pavement surface, and possibly some from the joint interface due to grinding action caused by slab movements.

Abrasion of the lower surface of the concrete slab is not considered a source of the buildup as in most instances the asphaltic curing seal was still intact on the slab bottom.

The effectiveness of load transfer across the undoweled transverse joints due to joint interlock was found to be highly variable with changes in pavement temperature. This would indicate that when the pavement top is cooler than the bottom, such as in the morning or during or after a rainstorm, the slabs are readily subject to deflection from heavy wheel loads.

When free water is available beneath the slabs in the vicinity of the joints, this water may be repeatedly moved in all directions under the downward deflection of the slabs caused by moving loads. As a load approaches a joint, water is moved in the leave direction relatively slowly, accumulating under the leave slab. As the load crosses the joint, there is a sudden rebound of the approach side of the joint creating suction, and a sudden depression of the leave side of the joint creating pressure, imparting to the accumulated water great force and therefore velocity in the direction of the approach side. As the wheel load continues past the joint, the water slowly returns to the leave side. The net effect is a series of low velocity movements of water in the leave direction and a series of high velocity water movement back toward the approach slab. The high velocity water movement would readily carry any available loose material backwards, depositing particles under the approach slab. This action, repeated over a period of time, eventually causes a buildup under the approach slab thus creating a "faulted" joint.

CONCLUSIONS

On the basis of this investigation, the following conclusions are warranted:

1. Faulting of PCC pavement joints is caused by an accumulation or buildup of loose material under the slabs near the joints. This accumulation may occur only under the approach slab, or may be a differential buildup under both slabs with the thicker layer under the approach side.

2. The buildup is caused by violent water action on available loose or erodable materials which are beneath or adjacent to the slabs. The water is moved backward and quite likely transversely as well, by the fast depression of the curled or warped leave slab under heavy wheel loads and by the suction caused by the release of the approach slab, eroding and transporting any loose material.

3. The major sources of the buildup are the untreated shoulder material and the surface layer of the cement treated base. Minor amounts may come from abrasion of the concrete joint interface and from material on the pavement surface moving downward through the joint.

In addition to the above, two other inferences could be drawn from this study:

1. The 13-year old Manteca and 17-year old Turlock projects were almost identical in environment, construction materials, traffic and design with the exception that at Turlock, the joints were sealed during construction and apparently kept sealed since, while the joints at Manteca were never sealed. Most of the joints at Manteca (13 years) were faulted more than 0.20-inch, while at Turlock (17 years), most were faulted less than 0.15-inch and riding quality remained good. Based on the evidence from these two projects alone, it would appear that joint sealing is an effective tool for retarding faulting. Unfortunately, similar projects for comparison and corroboration could not be found.

2. At Nipomo and Prosser where both unfaulted and faulted joints were opened, the compressive strengths of the

CTB in the unfaulted areas were much greater than in the faulted areas. At Nipomo, the CTB strength at the unfaulted joint was 640 psi, and 260 psi at the faulted joint. At the unfaulted and faulted joints at Prosser, the values were 1350 psi and 595 psi, respectively. The significance of the higher values at the unfaulted joints is believed to be in the improved erosion resistance provided by the higher strength CTB. It should not be assumed that high CTB strength in itself will eliminate faulting. In this study, the condition of the top one-half inch of the CTB seemed to be of great importance, and this would have very little effect on the measured strength of cores.

POSSIBLE SOLUTIONS TO PREVENT OR MINIMIZE FAULTING

1. Prevent or reduce slab curl. A curled slab not only leaves room for more free water, but deflects more under heavy wheel loads providing the force to move the water and loose materials. Unfortunately, a feasible method of constructing concrete pavement slabs without volume change propensities has not been developed. Four alternatives have been or are being considered to eliminate or reduce the curl:

   a. Continuous reinforcement. An undesirable feature of this method is the considerable increase in pavement cost.

   b. Use shorter joint spacing -- possibly 7 or 8 feet. While this would not eliminate curl, it would reduce the distance through which curl acts. It should also provide tighter joints with greater interlock. These desirable features are based on the premise that most of the joints could be induced to crack and "work".

   c. Construct base and pavement monolithically. Possibly the increased thickness and weight of a 1.15-ft. thick slab would reduce the curling tendencies with an attendant reduction in deflection.

   d. The use of dowels has also been considered. However, considerable controversy exists over whether or not they actually improve pavement performance. Past experience indicates that numerous problems may be encountered in doweled pavements. The use of a properly designed and installed dowel system might prolong the life of joints, but it is not believed that they are a permanent solution to joint curl or faulting.

2. Prevent or minimize the entrance of water. The area of prime concern is the pavement edge where separation of the asphaltic concrete shoulder from the pavement slab usually occurs. Some means of easily maintaining a seal at this joint would need to be developed. (Several states are already working at this.) Another possible aid would be a concrete shoulder built monolithically

and of the same thickness as the pavement, incorporating a longitudinal plastic ribbon joint between the pavement edge and shoulder. This should result in a relatively tight joint which would minimize the entrance of water and maximize the runoff to the outside.

The transverse joints should also be kept sealed. The use of plastic inserts for weakened plane joints shows great promise in that the reservoir created by sawing is eliminated. Most joints formed in this manner appear to be quite tight after over two years of service.

3. Provide free drainage to the outside for any water which finds its way under the slab to reduce the time water is available to erode base or shoulder materials. (This could actually be used as an alternative to "2" above.) A permeable asphalt treated material in the shoulder is being considered for this purpose.

4. Eliminate all loose material from under the slab and provide erosion resistant shoulders and base surfaces. (See Figure 32 which shows the present typical structural section.) The permeable asphalt treated shoulder proposed for drainage would probably function as erosion resistant material as well. Asphaltic and portland cement concrete shoulders (outside only) constructed full depth of the slab should also be tried.

   Since present CTB construction methods sometimes result in poor surface layers, modifications of these methods are being considered. Lean concrete or a plastic CTB as a base would likely provide better resistance to erosion than conventional CTB. The advantages and disadvantages of this alternative are being investigated.

In addition to these proposals, numerous other suggestions have been studied. The ones listed above, however, are considered to be the most likely to furnish a solution to the faulting problem at reasonable cost. While several experimental installations are planned, it is believed that a combination of Nos. 3 and 4 above will provide the most practical and economical results.

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A P P E N D I X

TEST RESULTS

Selection of Test Joints

At each test site at Sacramento, Vacaville, and Truckee, three joints were selected for moving and static wheel load tests. These joints represented either comparatively high, medium, or low load transfer effectiveness based on data obtained from at least 10 Benkelman beam deflection tests at each site. The load for these Benkelman beam tests was provided by the PCA semitrailer truck shown in Figure 19. During Benkelman beam tests, the single axle load was 18,130 lbs. and each dual-tire wheel load was 9065 lb. Pavement corner deflections were measured on both sides of the joint for static wheel load applications on each side of the joint.

Figure 19

Benkelman beam deflections, computed values of joint effectiveness, and joint openings are given in Table 1 for each of the three joints selected for further testing at the Sacramento, Vacaville, and Truckee sites. Deflections could not be measured at the inside lane test site at Vacaville because the median strip provided unsuitable beam support.

In general, joint effectiveness decreased as joint opening increased at each location. As the time of test varied between the locations, the values at one location should not be compared with those at another location. The effect of time of test on load deflection and joint effectiveness is shown by the results given in Table 2 from Benkelman beam tests conducted by State forces at Manteca and Turlock. Deflections measured in the late morning and afternoon were less than those measured in the early morning. Also, joint effectiveness values were greater in the afternoon when joint openings were usually smaller.

Exploratory Tests

Pavement deflections are a function of the degree of contact between the pavement slab and the subbase. This contact varies with slab curl due to temperature and moisture changes. When the top surface of a slab loses more moisture than the bottom surface, an upward movement of the slab edges occurs. When the top temperature is less than at the bottom surface, the relative shortening of the upper layer also causes upward movement of the slab edges. Conversely, as the temperature gradient reverses with greater top than bottom surface temperatures, the slab edges move downward from the upward curled position.

To determine typical corner curling values for the California pavements, exploratory measurements were made at the Sacramento test location. Temperature measured during the daylight hours are given in Figure 20 and corresponding changes in slab corner curl are given in Figure 21. It was assumed that the area of contact between slab and subbase was greatest when the corner curl measurements were minimum. This established zero datum. However, this does not indicate that the slab was in complete contact with the subbase at this time as pavement slabs are usually curled upward to some extent during the entire lifetime of the pavement.

A temperature increase also causes pavement slabs to expand and joint openings to close. Joint openings measured at joint No. 2 (see Table 1) at Sacramento are given in Figure 22.

TABLE 1

Benkelman Beam Test Results by PCA

Location	Joint			Defl. (0.001")		Eff. (%)	Time of Test	Joint Opening (0.001")
	Faulted	Skew	No.	Approach	Leave			
Sacramento	No	Yes	1 2 3	8 12 10	7 9 6	93 85 75	10:30 am	13 15 17
Vacaville (A)	Yes	Yes	1 2 3	16 20 25	13 11 8	90 71 49	9:30 am	28 40 58
Vacaville (B)	No	Yes	1I 2I 3I	-- -- --	-- -- --	-- -- --	8:00 am	15 25 40
Vacaville (C)	Yes	Yes	4 5 6	24 20 22	10 8 8	59 57 53	9:00 am	40 45 42
Truckee (Boca)	Yes	No	1 2 3	8 10 14	8 10 14	100 100 100	4:30 pm	20 25 32
Truckee (Prosser)	Yes	Yes	4 5 6	6 8 6	6 8 4	100 100 80	3:30 pm	25 43 51

TABLE 2

Benkelman Beam Results by State Forces

Location	Joint		Time of Test	Defl. (0.001")		Eff. (%)
	Faulted	Skew		Approach	Leave	
Turlock	Yes	No	8:00 am	31	14	62
			11:00 am	8	6	86
			1:45 pm	8	7	93
Manteca	Yes	No	9:00 am	11	2	31
			3:00 pm	3	3	100

AREA I - NEW PAVEMENT, SACRAMENTO

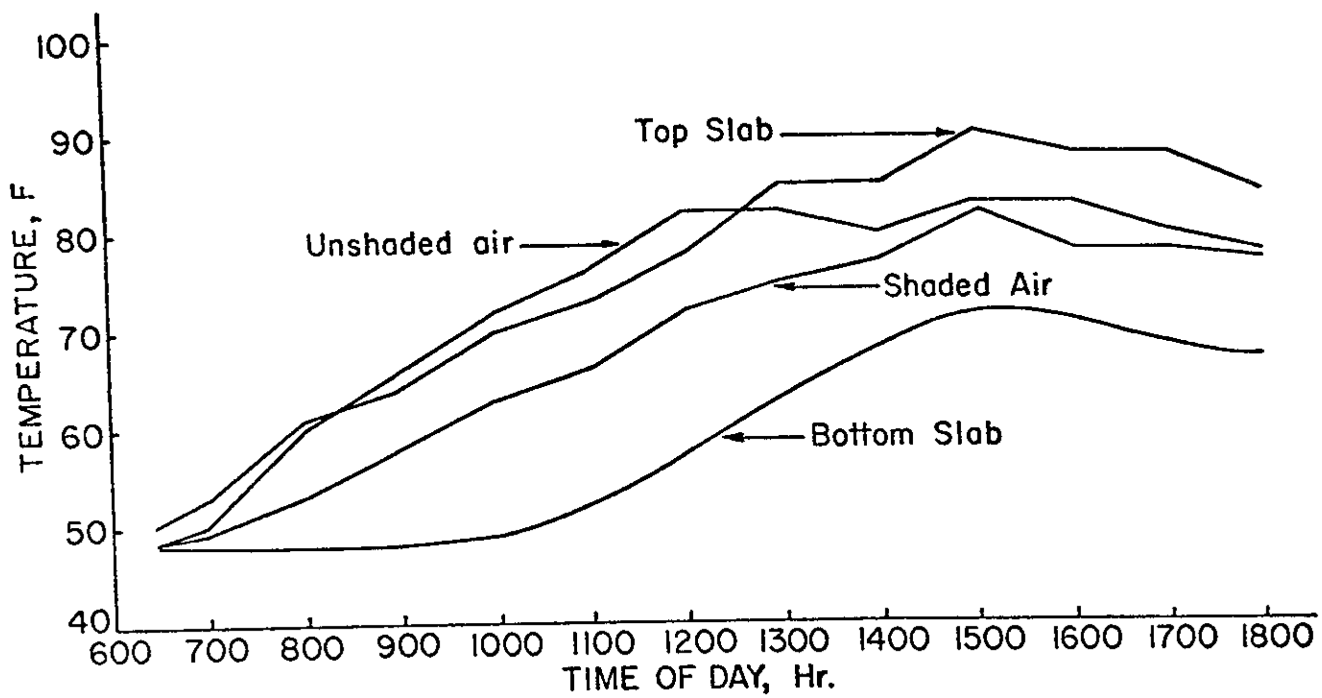


FIG.20- TEMPERATURE RELATIONS

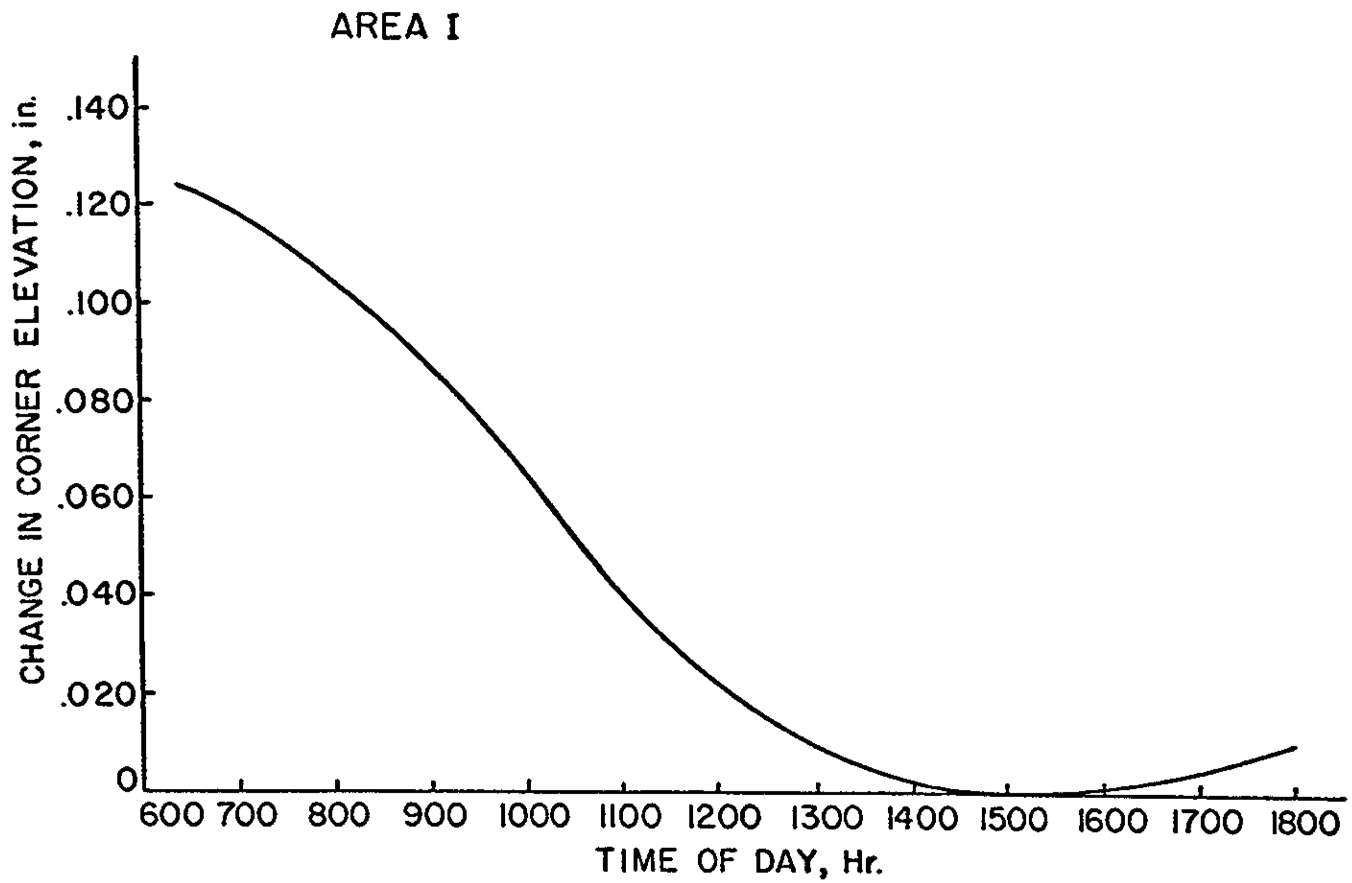


FIG.21- AVERAGE CORNER CURL

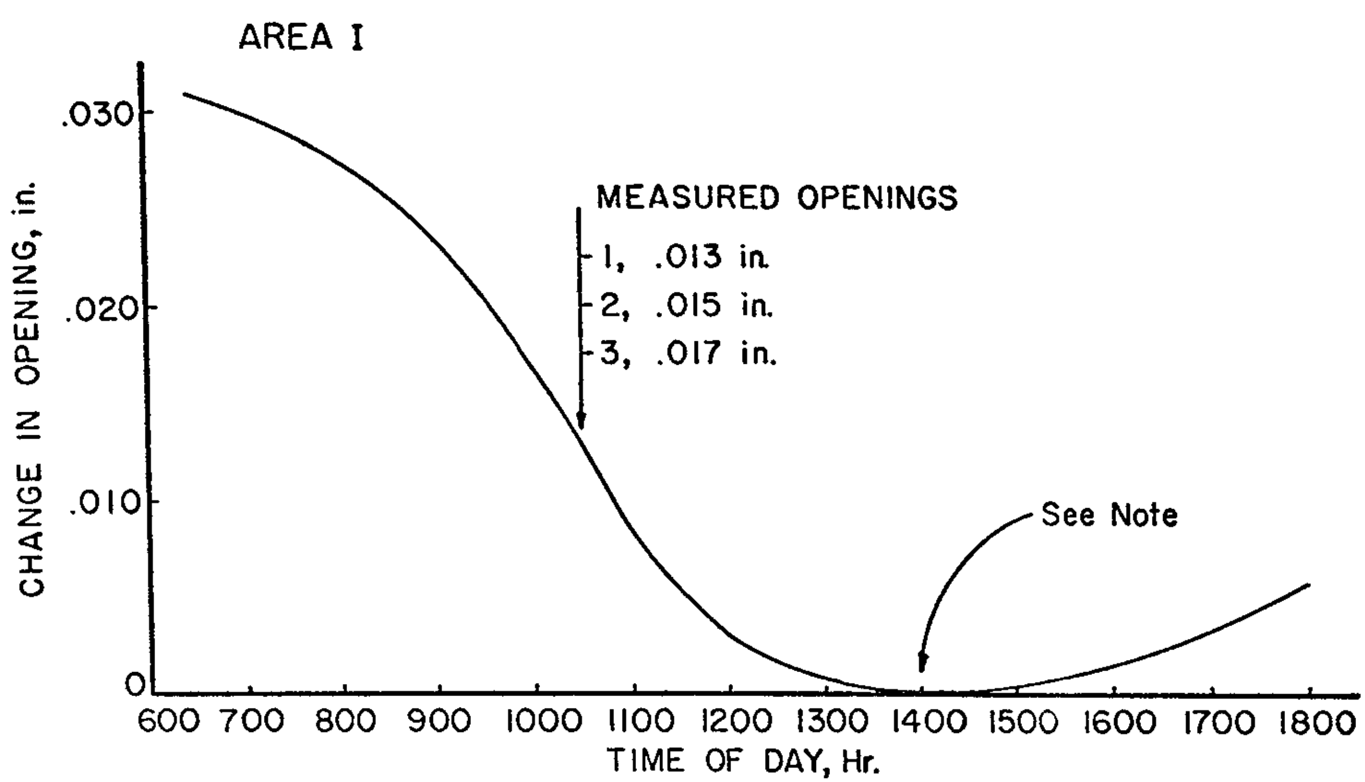


FIG.22 - JOINT OPENING

NOTE: Zero opening relative only to changes occurring during indicated time period.

As expected, the smallest joint opening occurred at approximately the same time that the slab top surface temperature no longer increased and the slab corners ceased to curl downward.

To investigate the effect of magnitude of corner curling on load deflections and strains, moving load tests were conducted on the new pavement at Sacramento during different times of the day between 7:00 am and 6:00 pm. Load was applied by the PCA truck moving at creep speed with the outer tire tread 2 inches from the pavement edge. The axle load was 18,130 lbs. Corner deflections were measured at the three joints selected for test and strains were measured by top surface gages at the longitudinal edge 5 feet from the joints. Average deflections from at least two load tests at each joint for each time period are given in Figure 23. Load deflections decreased from 0.035-inch at 7:00 am to 0.006-inch at 3:00 pm. It was shown in Figure 21 that upward corner curl decreased 0.120-inch between 6:30 am and 2:00 pm. Thus, early morning deflections were nearly six times those obtained in the afternoon, demonstrating the very significant influence of slab curling on the magnitude of corner load deflection. Edge load strains measured during these same tests ranged from 37 to 41 micro-inch per inch and showed no correlation with time of day.

The influence of wheel path on corner load deflection and edge strain was also investigated at Sacramento. These tests were conducted in the early morning at 7:00 am. The load truck was driven at creep speed with the outer tire tread 2, 10, and 26 inches inward from the pavement edge. Average deflections from the three test joints, as given in Figure 24, indicated that values in the 26-inch wheel path were 31 percent less than those in the 2-inch wheel path. Edge load strains measured during these same tests, as given in Figure 25, indicated that values in the 26-inch wheel path were 69 percent less than those in the 2-inch wheel path.

The influence of vehicle speed on corner load deflection and edge strain was also investigated at Sacramento. These tests were conducted at both 10:00 am and 2:00 pm. The load truck was driven at speeds up to 35 mph with the outer tire tread 10 inches inward from the pavement edge. Average deflections from the three test joints, as given in Figure 26, indicated that late morning load deflections decreased 43 percent as speed increased from 2 to 35 mph. By comparison, the early afternoon deflections decreased only 17 percent as speed increased from 2 to 15 mph and then remained unaffected by additional increase in speed. Edge load strains measured during these same tests, as given in Figure 27, indicated that strains decreased 30 percent as speed increased from 2 to 35 mph. As previously stated, strains measured in the early

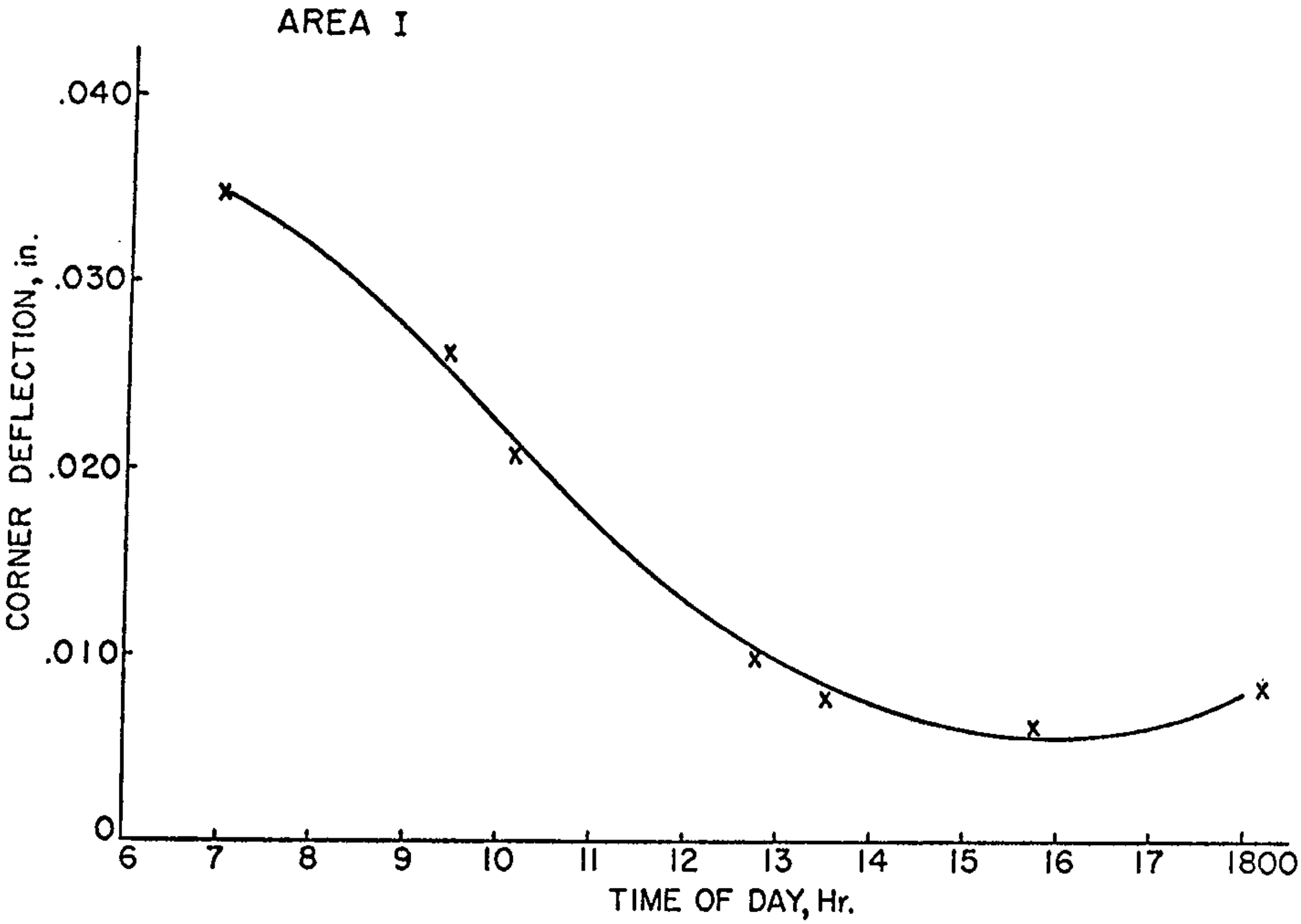


FIG.23-EFFECT OF CORNER CURL ON LOAD DEFLECTION
(CREEP SPEED)

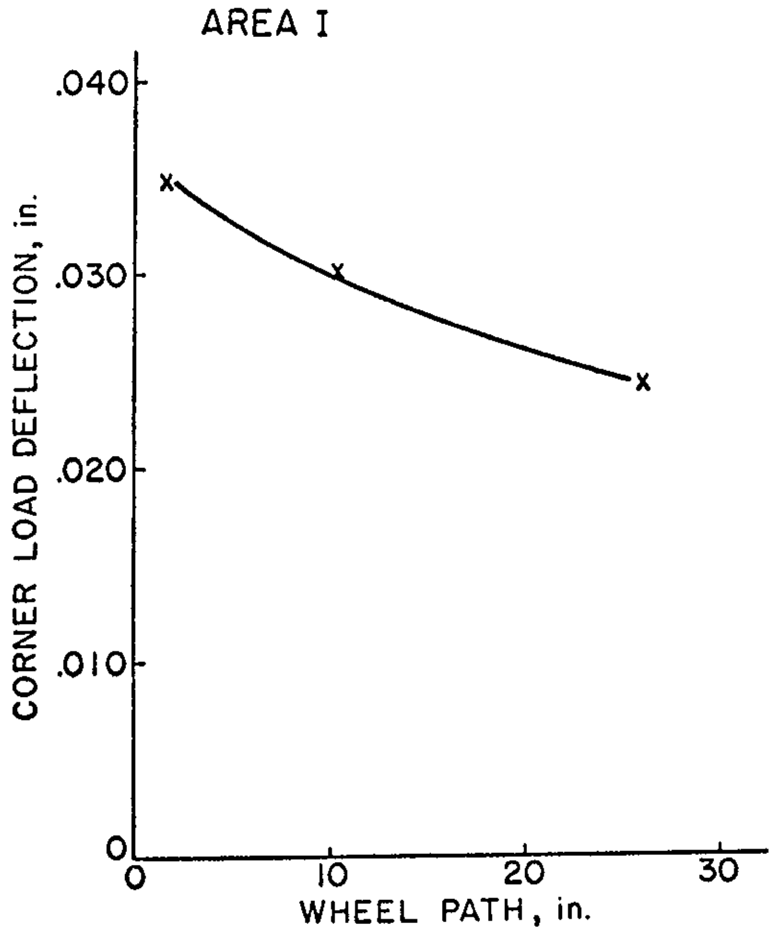


FIG.24- EFFECT OF WHEEL PATH ON CORNER LOAD DEFLECTION
(CREEP SPEED - EARLY A.M.)

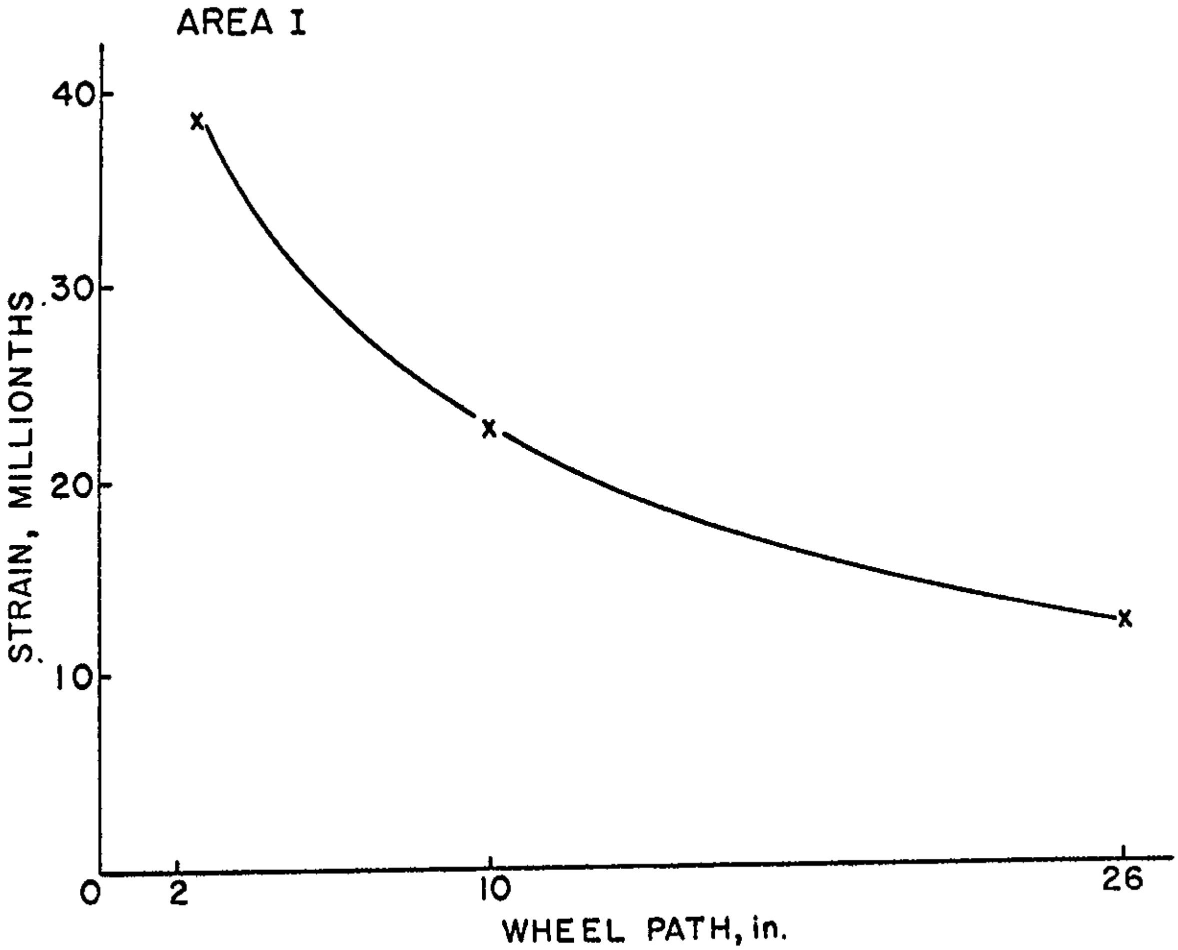


FIG.25- EFFECT OF WHEEL PATH ON AVG. EDGE STRAIN (CREEP SPEED)

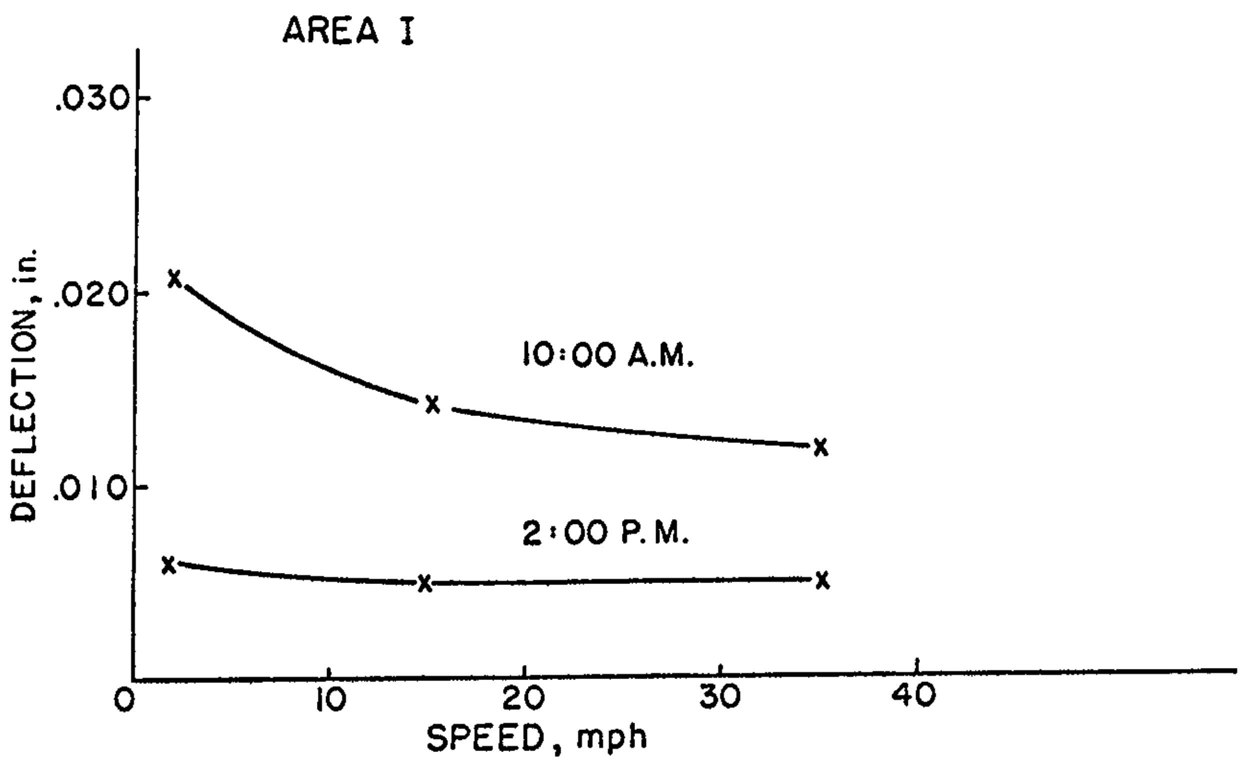


FIG.26- EFFECT OF SPEED ON LOAD DEFLECTION
(NEW PAVEMENT; 10-in. WHEEL PATH)

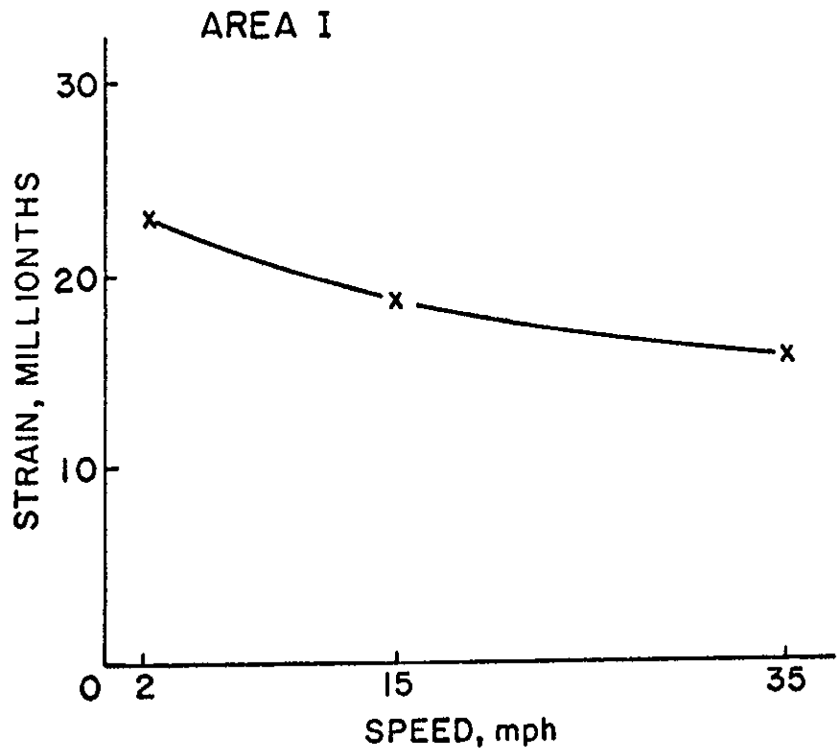


FIG.27- EFFECT OF SPEED ON LOAD STRAIN
(10-in. WHEEL PATH, LATE P.M.)

afternoon were similar to strains measured in the late morning.

A special load test was also conducted at Sacramento to study the influence of faulting on load deflection at slab corners. For this purpose, a 1/2-inch thick board was placed adjacent and parallel to a joint on the side from which the load approaches. Thus, a passing wheel load crossing the joint would drop 1/2-inch from the board to the pavement. Load was applied by the PCA truck moving at creep speed with the outer tire tread a distance of 26 inches from the pavement edge. Tests were conducted in the morning when differences in results would be enhanced by greater magnitudes of deflection. The corner deflection was 0.019-inch for the case of a 1/2-inch fault compared with 0.016-inch when the 1/2-inch thick board was removed. Therefore, corner deflections may be slightly greater if a joint is faulted.

Load-Deflection Tests

To compare load deflections at Sacramento, Vacaville, and Truckee, loads were applied at each location by the PCA truck moving at creep speed with the outer tire tread a distance of 2 inches from the pavement edge. Tests were conducted in the afternoon when all slabs were in the least curled condition. Load deflections measured at the corners on each side of the joints selected for test were averaged for each test location. These average deflections, as given in Table 3, ranged from 0.004 to 0.008-inch.

Load Strain Tests

Static load tests were conducted at Sacramento and Truckee to determine load-strain relationships at skewed joints. The PCA truck was used to apply a static wheel load at either of two positions. In one position, both truck wheels were placed on the approach slab as close to the joint as possible. In the other position, the truck wheels were placed on the leave slab as close to the joint as possible. Strain gages were placed along the corner bisector of both the approach and leave slab corners as illustrated in Figure 28. Load-strain results for each load position at Sacramento are shown in Figure 29. Maximum strain was located approximately 5 and 4 feet from the apex of the approach and leave corner respectively. In the afternoon when the slab was less curled than in the morning, maximum tensile strains on the corner bisector due to corner load were less than in the morning period. Maximum strains in the morning were approximately 28 micro-inches per inch

TABLE 3
Load Deflections

Location	Joint		Average Deflection (0.001-in.)
	Faulted	Skew	
Sacramento	No	Yes	6
Vacaville (1 and 3)	Yes	Yes	8
Vacaville (2)	No	Yes	8
Truckee (1)	Yes	No	4
Truckee (2)	Yes	Yes	8

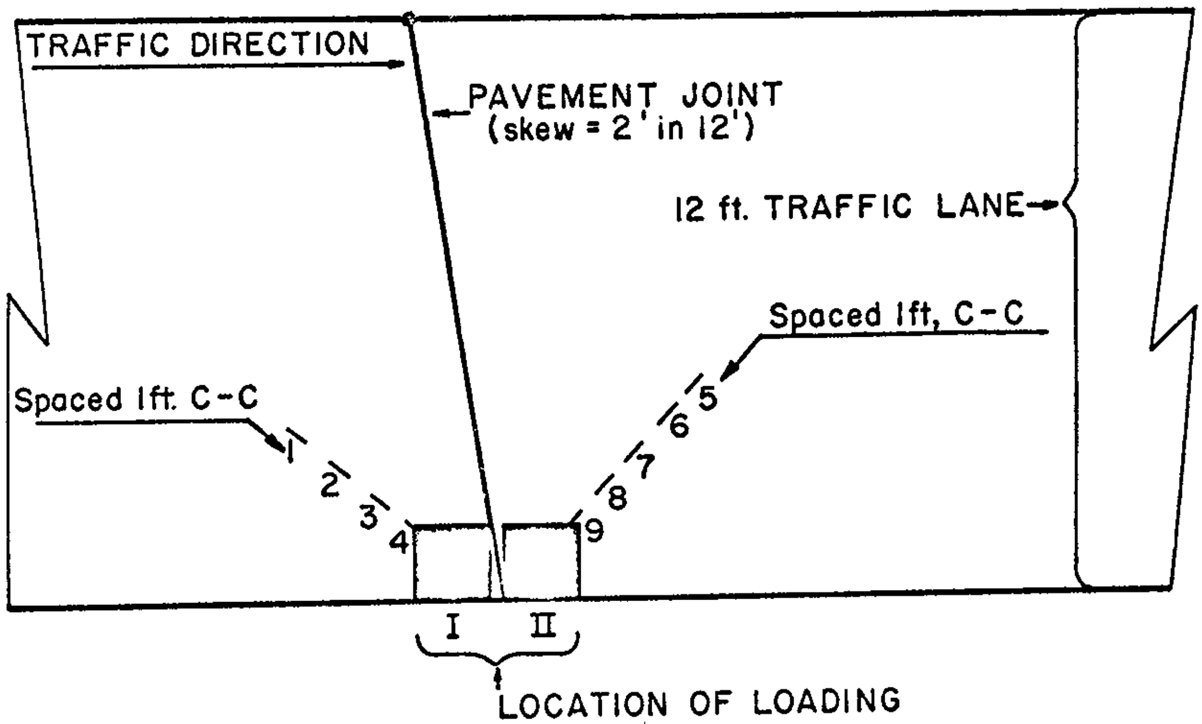


FIG.28- ARRAY OF STRAIN GAGES - AREA I

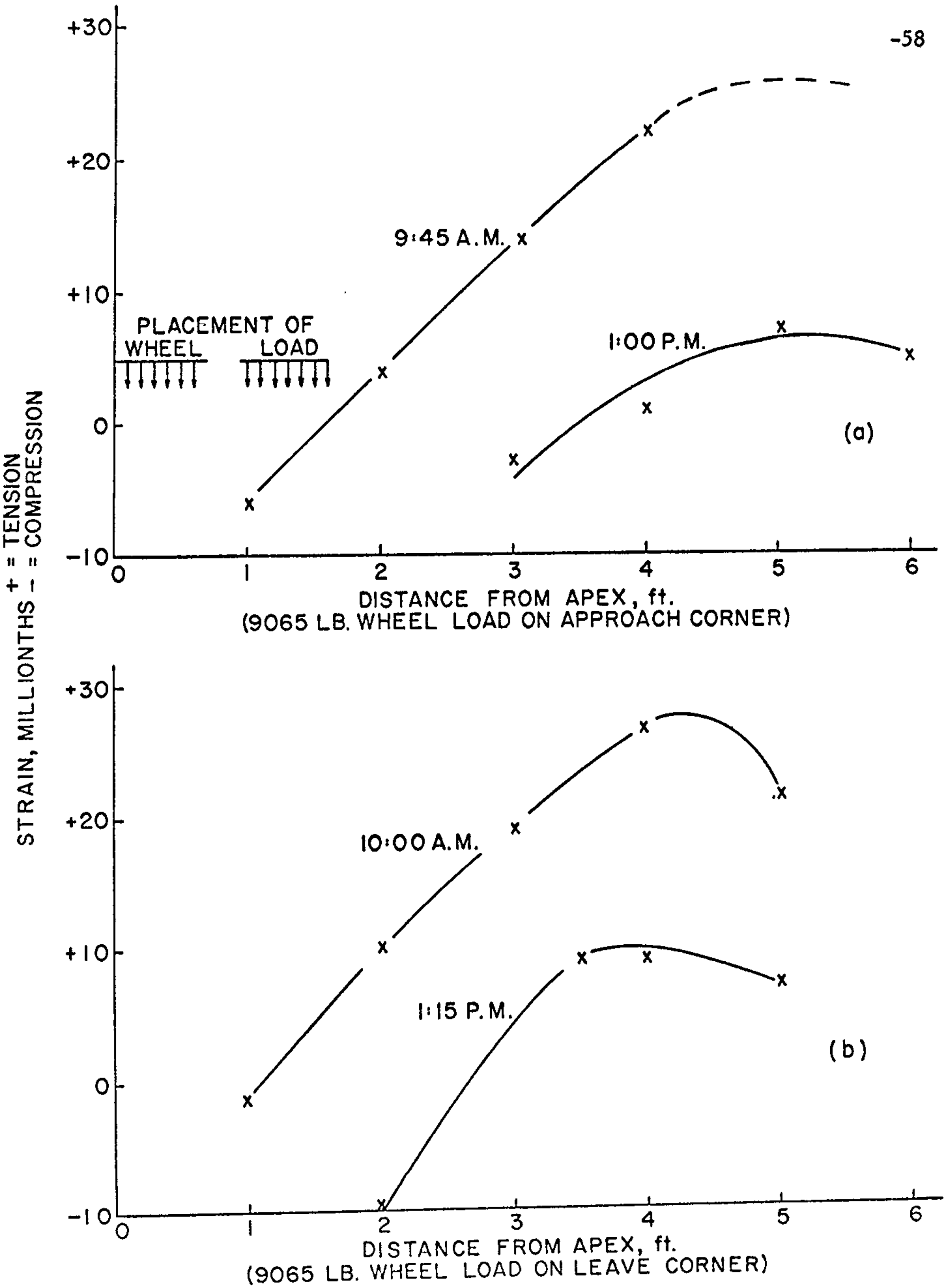


FIG.29- CORNER LOAD STRAIN ALONG CORNER BISECTOR

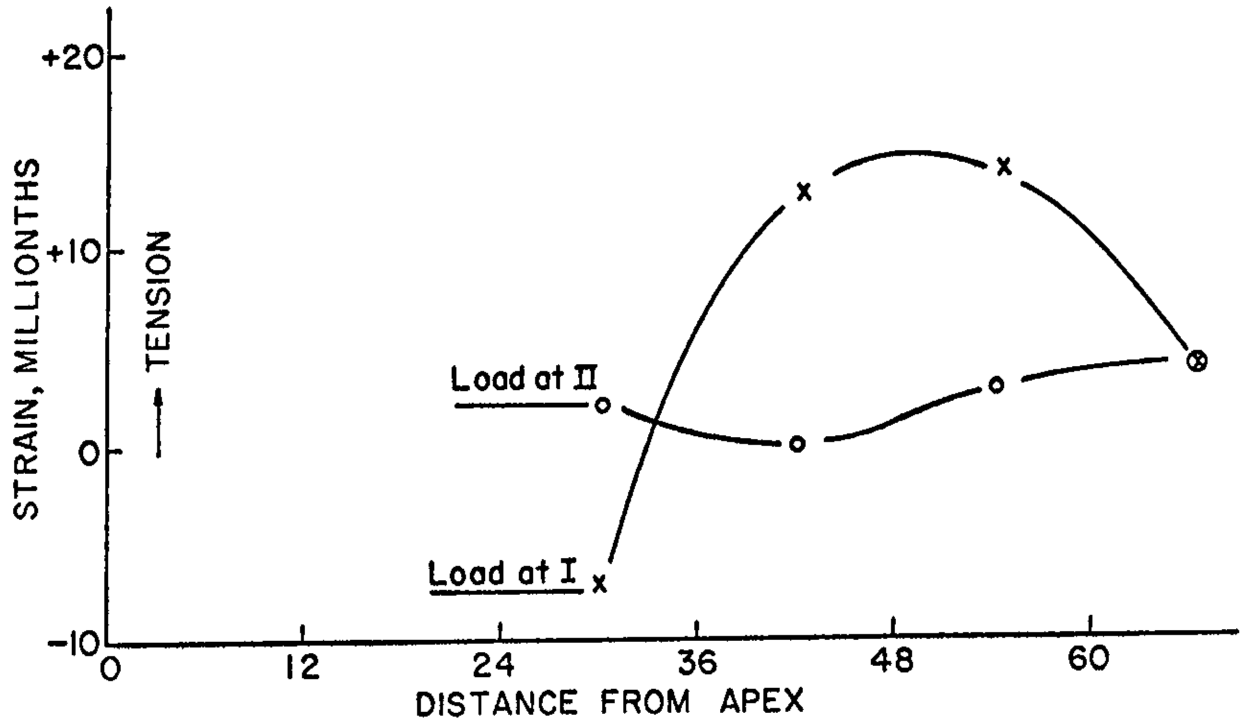


FIG.30- LOAD STRAIN ALONG CORNER BISECTOR

and maximum strains in the afternoon were approximately 10 micro-inch per inch.

At Truckee, load tests were conducted at 10:00 am on the approach slab and at 1:00 pm on the leave slab. Load-strain results for each load position at Truckee are shown in Figure 30. Maximum strain was located approximately 4 and 5 feet from the apex of the approach and leave corner, respectively. Maximum strains in the morning on the approach slab were approximately 15 micro-inch per inch and maximum strains in the afternoon on the leave slab were approximately 5 micro-inch per inch. These values are about one-half of the maximum strains measured at Sacramento.

After pavement slabs and subbase materials were removed at Sacramento and Vacaville, 30-inch diameter plate bearing tests were made on the untreated base material beneath the CTB on each side of the joint to determine if differential k-values had resulted from slab loading. The results, as given in Table 4, indicate that k-values were approximately the same on both sides of the joints.

Summary

1. Measurements made at the Sacramento test location indicated that between 6:30 am and 2:30 pm, the air temperature increased 32°F, while the average corner elevation decreased 0.120-inch and the joint opening decreased 0.030-inch.

2. Benkelman beam tests indicated that joint effectiveness generally decreased as joint opening increased.

3. Corner load deflections at Sacramento due to an 18,130-lb. axle load decreased from 0.035-inch at 7:00 am to 0.006-inch at 3:00 pm. Edge load strains measured during the same time period ranged from 37 to 51 micro-inch per inch and showed no correlation with time of day.

4. Corner load deflections at Sacramento were 31 percent less when the outer tire tread was 26 inches instead of 2 inches from the pavement edge. Edge load strains were 69 percent less when the outer tire tread was 26 inches instead of 2 inches from the pavement edge.

5. Corner load deflections measured in the late morning at Sacramento decreased 43 percent as speed increased from 2 to 35 mph. By comparison, early afternoon deflections decreased 17 percent as speed increased from 2 to 35 mph. Edge load strains decreased 30 percent as speed increased from 2 to 35 mph.

6. Corner load deflections were measured at Sacramento, Vacaville, and Truckee in the afternoon when slabs were in the least curled conditions. Average deflections at each location ranged from 0.004 to 0.008-inch.

7. Load strains were measured at Sacramento and Truckee along the corner bisector of both the approach and leave slab corners. Maximum strains occurred between 4 and 5 feet from the apex of the corner. Maximum strains were 28 and 15 micro-inch per inch at Sacramento and Truckee, respectively.

8. On the basis of the limited tests, it appears that deflections and strains are not lessened by the use of skewed in lieu of right angle joints. However, there is no apparent correlation between deflections and strains at joints and faulting.

TABLE 4

Plate Bearing Tests on Untreated Base Material

Location	Joint		k-value (pci)	
	Faulted	Skew	Approach	Leave
Sacramento	No	Yes	356	330
Vacaville (1)	Yes	Yes	334	321
Vacaville (2)	No	Yes	250	252
Vacaville (3)	Yes	Yes	288	260

Rainfall (or Equivalent) After Tracer
Sands Placed
(March - May, 1969)

San Luis Obispo - Santa Maria Area					
Date	Inches	Date	Inches	Date	Inches
March	0	Apr. 2 6 8 9 22 23	0.64 1.83 0.04 0.10 0.25 0.06	May	0
Truckee - Boca Area					
Mar. 30	0.08	Apr. 5 6 14 18 23 24	0.32 0.36 0.04 (snow) 0.11 0.12 (snow) 0.97 (snow)	May 4 10 11	0.30 0.25 0.14

Physical Data - 1968 Investigation

No.	Location	Age, Yrs.	Joint					Avg. Compressive Strength		One-Way Traffic	
			Faulting, Inches	Spacing, Feet	Skew	Seal	Movement[1] Inches	P.C.C.	C.T.B.	ADT	ADTT
1	Sacramento	1/4	0	13, 19, 18, 12	Yes	No	0.031	6400	800	0	0
2A	Vacaville	5	0.07 to 0.25	13, 19, 18, 12	Yes	No		5500	1100		
B								6300	----	17000	2200
C								6600	----		
3A	Truckee-Boca	9	0 to 0.15	15	No			6300			
						Yes			----	5000	350
B	Truckee-Prosser		0 to 0.20		Yes			6300			
4	Manteca	13	0.10 to 0.30	15	No	No	0.022	5600	550	9000	1800
5	Turlock	17	0.10 to 0.20	15	No	Yes	0.029	5200	310	12000	1900

[1]Average of 3 joints, measured from early morning to hottest part of the day.

Physical Data - 1969 Investigation

No.	Location	Age, Yrs.	Joints: Faulting, Inches	Joints: Spacing, Feet	Joints: Skew	Joints: Seal	Joints: Width[1] Inches	Joints: Movement[2] Inches	Avg. Compr. Str. P.C.C.	Avg. Compr. Str. C.T.B.	One Way Traffic ADT	One Way Traffic ADTT
1	Pismo Beach	13	0.15 to 0.30	15	No	No	-----	0.014	6300	*	7800	700
2	Russell Turn	13	0.15 to 0.30	15	No	No	0.011	0.015	4900	220	7000	700
3u 3f	(Unfaulted) Nipomo (Faulted)	13	0.10 to 0.30	15	No	No	0.020	0.007	4900	640 260	7500	700
4u 4f	Prosser	10	0 to 0.15	15	Yes	Yes	0.011	0.003	5600	1350 595	5000	350
5u 5f	Yuba Gap	6	0 to 0.20	13 19 18 12	Yes	Yes	0.023	0.009	5700	* *	5000	350
6	Boca	10	0 to 0.15	15	Yes	Yes	-----	-----	6800	*	5000	350

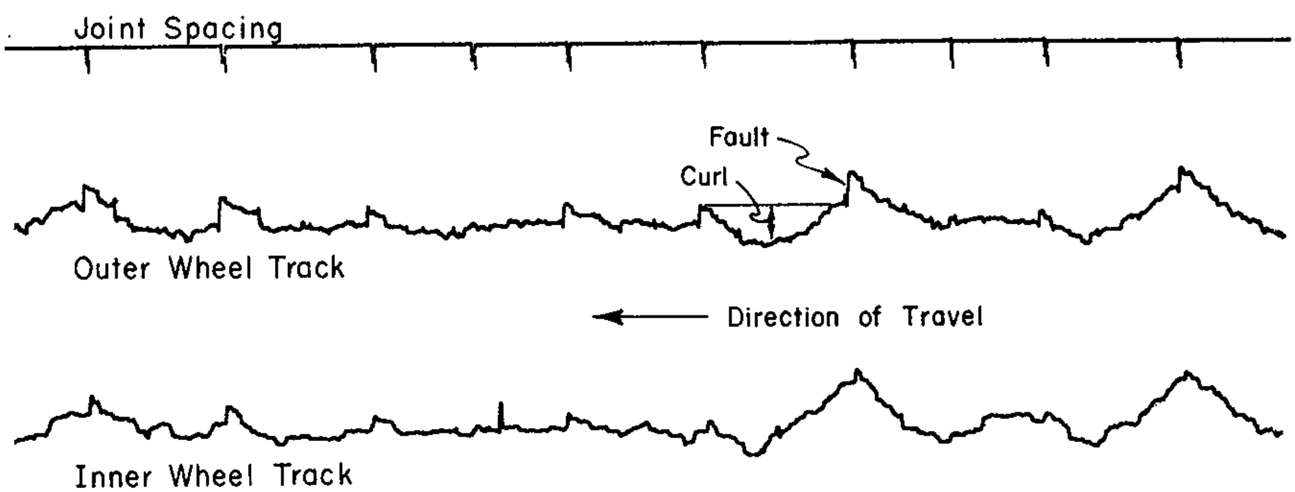
[1]Average of 3 joints, measured at closest points at hottest part of day.

[2]Average of at least 3 joints, measured from early morning to hottest part of day.

*CTB was so weak, cores could not be obtained.

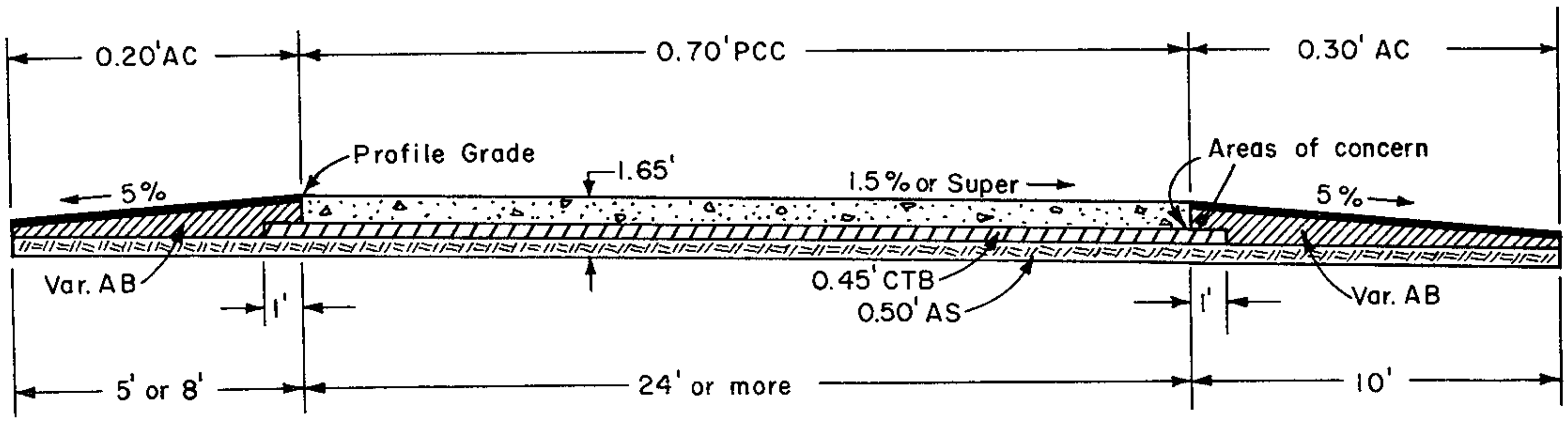
Figure 31

TYPICAL PROFILOGRAM - VACAVILLE AREA



SCALE
VERT. 1" = 1"
HORZ. 1" = 25'

Figure 32



TYPICAL HALF SECTION

SYMBOLS & ABBREVIATIONS

PCC	Portland Cement Concrete.
AC	Asphalt Concrete–Type B (unless otherwise noted).
CTB	Cement Treated Base–Class B.
AB	Aggregate Base – Class 2.
AS	Aggregate Subbase–Class 4.